Appendix H. Levee Stability and Seepage Technical Report

SUPPLEMENTAL GEOTECHNICAL REPORT

LEVEE STABILITY AND SEEPAGE ANALYSIS REPORT FOR THE DELTA WETLANDS PROJECT REVISED EIR/EIS

Prepared for
Jones & Stokes Associates, Inc.
2600 V Street
Sacramento, California 95818

May 22, 2000

URS Greiner Woodward Clyde
500 12th Street, Suite 200
Oakland, California 94607

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URS

May 22, 2000

Ms. Aimee Dour-Smith
Jones & Stokes Associates, Inc.
2600 V Street,
Sacramento, CA 95818
Ph. (916) 737-3000

Subject: Supplemental Geotechnical Report
Levee Stability and Seepage Analysis Report
for the Delta Wetlands Project Revised EIR/EIS

Dear Ms. Dour-Smith:

We have completed our report for the geotechnical evaluation of the proposed storage islands (Bacon Island and Webb Tract) for the Delta Wetlands Project as described in your draft EIR/EIS dated September 1995. The work included in this report has been conducted in accordance with our revised scope of work dated September 13, 1999. This report also incorporates responses to comments by the State Water Resources Control Board received on our draft report dated December 15, 1999, and the second draft dated March 17, 2000.

The report includes our evaluation of the geotechnical issues and concerns raised in the State Water Resources Control Board's letter dated November 25, 1998 in regards to the draft EIR/EIS report. The findings and conclusions from our evaluation for the seepage issues are presented in Section 2 of this report, and those from stability and settlement evaluations are included in Section 3. In addition, a summary of the key findings for all aspect of our evaluation is included in Section 4.

The work included in this report was conducted in accordance with URS quality assurance plan. Dr. Ulrich Luscher was the senior technical review officer.

We will be pleased and available to provide any clarification, explanation or further details on the contents of this report.

Sincerely,

URS Greiner Woodward-Clyde

Said Salah-Mars, Ph.D., P.E.
Senior Project Manager

Ulrich Luscher, Ph.D., P.E., G.E.
Senior Consultant

URS Corporation
500 12th Street, Suite 200
Oakland, CA 94607-4014
Tel: 510.893.3600
Fax: 510.874.3268

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PURPOSE OF SUPPLEMENTAL GEOTECHNICAL STUDY

The primary purpose of this supplemental geotechnical evaluation of the Delta Wetlands Project is to address concerns expressed by the State Water Resources Control Board in a letter dated November 25, 1998. The letter raised a number of questions related to geotechnical issues included in the Draft EIR/EIS (Jones & Stokes, 1995). Specific issues raised in Section III of Attachment A to the SWRCB letter included several aspects of seepage, seepage control by interceptor wells, and seepage monitoring; and levee stability aspects. A decision was then made that a supplemental EIR/EIS (referred to as a Revised EIR/EIS or REIR/EIS) for the project should address these issues and provide more detailed evaluation of the geotechnical issues of the project. Jones & Stokes Associates engaged URS Greiner Woodward Clyde (URSGWC) to provide the supplemental geotechnical evaluations and prepare a supplemental geotechnical report. The present report is the result of this work.

PROJECT UNDERSTANDING AND BACKGROUND

The objective of the Delta Wetlands (DW) Project is to provide water storage in the Sacramento/San Joaquin River Delta. The project plans to store excess runoff water during heavy winter and spring runoff on two Delta islands and release the water later in the year for beneficial use and water supply. The planned reservoir islands are Bacon Island and Webb Tract. In addition, two islands are planned to be converted to wetlands for environmental mitigation; these islands are Bouldin Island and Holland Tract. The project is fully described in the Draft EIR/EIS prepared by Jones & Stokes Associates (1995).

Conceptual plans for the conversion of the reservoir islands from agricultural use to water storage use have been developed by DW and are also described in the Draft EIR/EIS. A brief summary of the proposed concept is provided below.

The levees around the reservoir islands will be raised and strengthened, such that the islands could store water up to a maximum elevation of +6 feet (all elevations are related to NGVD). Erosion protection on the levees will be raised on the channel side and will also be provided on the reservoir side. Siphons with supplemental pumps will be installed to fill the reservoir islands during periods of surplus flows through the Delta. Pumps will move the stored water back into the Delta waterways when it is needed by the users. The reservoir islands could be completely emptied by pumping.

A system of a large number of extraction wells installed on the levees of the reservoir islands has been proposed by DW to protect the islands adjacent to the reservoir islands from the anticipated effects of seepage from the reservoir islands. Such seepage effects are expected because of a deep sand aquifer that underlies the reservoir and the adjacent islands as well as the channel or slough separating them. To control the amount and duration of pumping from these wells that the effect on the adjacent islands is small or insignificant, DW has proposed a network of monitoring wells. This network would include both seepage monitoring wells immediately across the channel from the reservoir islands and background wells to establish water level changes that would occur without water storage on the reservoir islands. DW has also proposed significance criteria that will provide the method by which monitoring well data are used to control pumping of the extraction wells.

ASSUMPTIONS AND LEVEL OF EFFORT

The scope of the work presented in this report was to review the geotechnical aspects of the Draft EIR/EIS related to seepage and stability, and perform an independent evaluation of the completeness and adequacy of the findings and conclusions. In general the work performed for the Draft EIR/EIS was presented at a conceptual level typically commensurate with environmental impact statements. Similarly our independent evaluation has been developed at a comparable conceptual level, and hence no detailed engineering and design are intended to be part of this study. The level of detail involved in our analyses is based on existing information developed by others in previous studies.

No additional investigations or testing programs were conducted for this work. Site-specific investigations and testing programs were not part of the scope of this evaluation. The levees' properties, subsurface soil conditions, seismic setting and hydraulic and hydrologic conditions were therefore characterized based on available data, publications, and engineering judgement and experience. Because of the size of the project (over 30 miles of levees) and the number of extraction and monitoring wells, the characterization of the site-specific subsurface conditions at each and every well or at every mile post of levee is beyond the scope of this work. The existing data, although limited in scope for design purposes, are nonetheless useable for a feasibility-level evaluation.

Where previous work has been done from a reliable source, such as the seismic vulnerability work performed by the CALFED committee, our seismic stability analysis was built on the findings from that work. The use of available levee and subsurface data was optimized by locating cross sections and/or profiles for analysis where the most information was available. The analyses are hence performed for values within the boundaries of the data ranges available at the site. Assumptions of extreme lower or higher values outside the range of available data were not considered in the analyses.

The principal approach used in the evaluation of the project impacts was to identify the relative incremental changes of the proposed project to the "without-project" condition (baseline case). The analytical models developed for the existing conditions (baseline or "without-project" condition) concentrated on calibrating the aquifer properties such that the groundwater levels inside the islands are matched given the levee geometry and water level in the sloughs. The project impacts were assessed by allowing the project criteria, such as the reservoir water level and the levee configuration, to change while maintaining constant the inherent parameters of the baseline condition. For example, for the seepage control measures, a parametric variation was applied to the extraction wells' pumping rate until no significant change in the neighboring islands was observed.

Our evaluations were made on two cross sections on each proposed reservoir island. These cross sections, which were different for seepage and stability evaluations, were selected based on available data to be reasonably representative but on the conservative side for seepage and stability issues, respectively. The most severe conditions that may be encountered may not have been considered. Nevertheless, the results for the sections that were analyzed suggested in all cases that more severe conditions could be accommodated with suitable changes in the design. Such accommodation will need to be considered in the final design.

EVALUATION AND FINDINGS

The work presented in this report can be defined along three main aspects. One aspect addresses the seepage issues and extraction wells operation, the second aspect addresses the significance criteria, and the third aspect addresses the levee stability condition.

Seepage Evaluation

To evaluate the project-induced seepage impacts on the neighboring islands and the proposed seepage mitigation, we have developed a two-step approach. First we built a seepage model that represents the baseline conditions (without-project) and calibrated the model against the observed conditions. Specifically, the levee and subsurface conditions and geologic profiles were developed using exiting boring log data. The monitoring well data were used to define the ground water condition within the project islands. The surface water levels in the drainage ditch were used to establish the data along the surface drainage system. The water levels recorded in the nearby gauging stations, within the Delta, were used to set the water levels in the surrounding sloughs. Empirical correlation relationships between grain size distribution and soil permeability were developed from available grain size distribution curves for the various soil strata and available permeability tests. Except for the potential variation of the permeability values, the rest of the data was relatively anchored into soil test results or water level readings. We have consequently calibrated the model by allowing the permeability values to vary until conditions similar to the baseline case are matched.

In a second phase, we built the "with-project" seepage model to assess the impact on the neighboring islands as a result of filling the reservoir islands. The outcome of the analysis focused on evaluating the impacts of the reservoir filling and the new levee configuration on the changes in the hydraulic head, the exit gradients (hydraulic gradient just below the ground surface), the flow rates, and the groundwater level in the neighboring islands. Based on the observed changes, the pumping rate and well spacing were varied consecutively until the baseline conditions "without-project" were restored. This exercise was repeated for a range of permeability values of the aquifer as supported by the soil test data.

For the borrow site, we used the same seepage model and added a borrow pit at various distances from the levee to estimate the minimum distance to the levee beyond which no impact to the above parameters was observed.

The findings from the seepage analysis were based on two representative cross sections for each island. The cross sections at each island were selected for the "narrowest" and "widest" slough width across a reservoir island and its neighboring islands. These cross sections represent somewhat a bounding of the seepage conditions. Below is a summary of our evaluation and findings.

- The proposed reservoir islands will have undesirable seepage flooding effects on adjacent islands if seepage mitigation measures are not used.
- Seepage control by extraction wells placed on the levees of the reservoir islands, as proposed by DW, appears effective to control undesirable seepage effects. Required well spacing and pumping rates appear to be manageable.

- A system of checking the performance of individual wells and of well maintenance needs to be developed and implemented. Well maintenance should be documented and tracked, to identify wells requiring excessive maintenance and potential adverse de-silting of the aquifer.
- The seepage analysis also indicates that the seepage flow from the nearby sloughs is not significantly different from the flows that occur currently into the islands without the project. Further, the percentage of the pumped flow originating from the slough side is at most 8% of the total pumped flow when the reservoir is full.
- Operation of the reservoir islands will lead to only small additional settlements, smaller than the settlements that the islands would experience with continued use as farmland.
- Wind-induced waves during reservoir operation are expected to be significant enough to require scour and erosion protection of the inner levee slopes.
- A minimum of 800 to 1,000 feet offset from the levee toe should be maintained for the location of borrow sites. With this offset, there is no discernible effect of the borrow areas on seepage.
- The sensitivity analysis considered the channel silt permeability, aquifer permeability, and the thickness of the peat layer within the reservoir island. The results indicated that the permeability of the channel silt and the aquifer have a significant impact on the seepage conditions and pumping volume, while the peat thickness has little effect.

Significance Criteria

DW proposed a seepage monitoring system to identify potential adverse impacts on the neighboring islands due to the implementation of the project. Significance standards were proposed by DW to evaluate when the seepage monitoring data would require initiation of seepage control measures. The work performed in this study consisted of reviewing the proposed seepage monitoring system, the historic water level data, the significance standards, and the seepage control measures. Further, an evaluation of the adequacy of the proposed seepage monitoring system and the significance standards was conducted.

The data collected from existing monitoring wells over the past 10 years are proposed as the "historic" conditions around which the significance criteria were developed. DW proposes to install a network of monitoring wells (piezometers) in the neighboring islands to provide seepage data during project implementation. In addition, background wells (far from the reservoir islands) are proposed to be used as future baseline data. During filling and storage, data from monitoring wells on neighboring islands will be compared to the historical and background data. The purpose of the comparison with historic data is to evaluate whether a correlation exists between the piezometric levels and the reservoir filling and storage. The comparison with the background data is to check whether deviations from historic data are occurring throughout the Delta or only near the reservoir islands. Below is a summary of our evaluation and findings.

- The need for monitoring and maintaining compliance with significance criteria is essential and must be carefully adopted and implemented.

- Use of seepage monitoring wells, as proposed by DW, appears suitable and reasonable. The number of background wells should be such that enough redundancy is available at each row of monitoring wells (piezometers) within the neighboring islands.
- Background wells should include both those conceptually proposed by DW and additional rows of shallow background wells across each adjacent island.
- Well readings by means of automatic data acquisition is appropriate and necessary for rapid response.
- Significance criteria have been developed by DW in consultation with others to apply the monitoring results to trigger seepage mitigation, consisting in the first place in pumping from the interceptor wells. The concept and format of the significance criteria appear appropriate, but some changes in the criteria appear desirable.
- The significance criteria should be reevaluated and updated periodically.

Stability Analysis

The stability of the project's levees has been evaluated by extensive stability analyses of sections selected to be representative of the more severe stability situations expected at the reservoir islands. The calculated factors of safety have been compared to various published stability criteria, and judgments were made of the adequacy of the proposed project in regard to levee stability.

For the seismic performance of the levees, two horizontal earthquake acceleration time histories recorded during past earthquakes were selected as the base motions for the analysis. These records were from the 1992 Landers and the 1987 Whittier Narrows earthquakes. The selected acceleration time histories were then modified to match the "design" earthquake response spectrum. Results from the recent CALFED study on the seismic hazard and levee failure probability of the Delta project were used to construct the "design" response spectrum . A hazard exposure level corresponding to a 10% probability of exceedance in 50 years was selected as "design" basis ground motions. This hazard exposure level results in a return period of about 475 years and is consistent with the requirement adopted by the 1997 Uniform Building Code (UBC).

For the assessment of geologic hazards, two earthquake design criteria were used: earthquakes with magnitude (M_w) 6 and peak ground acceleration of 0.25g, and magnitude (M_w) 7.7 and peak ground acceleration of 0.13g. These ground motions represent the local and distant controlling seismic events and are consistent with the results of the CALFED study (CALFED, 1999).

The resulting conclusions and recommendations are:

- The levee strengthening measures conceptually proposed by DW are generally appropriate and adequate to provide stability of the reservoir islands' levees, except as noted below.
- Similarly, the seepage monitoring and control measures are generally adequate to avoid reducing the stability of adjacent islands' levees, provided the recommended measures are implemented.

- Construction of the levee strengthening fills must be implemented in a manner to prevent stability failures due to the new fill loads. This will require carefully planned, staged construction, and monitoring to observe the behavior as the fill is placed. The staged construction will require a construction period estimated to extend over 4 to 6 years, depending on final design.
- Long-term stability toward the slough side will be reduced by the construction and reservoir filling to an excessive degree. Measures should be provided to improve this stability. Some conceptual slope stabilization measures may include: 1) flattening the slough side levee slope, 2) widening of the levee crest to provide redundant levee width, 3) rock buttressing the levee toe on the slough side. The environmental impacts of slope failure are not part of the scope of this work.
- Stability with respect to sudden drawdown of the water in the reservoir may be inadequate at some locations. This potential failure mode can be remediated by controlling the reservoir lowering, flattening the levee slopes, and armoring the slope faces.
- The seismic stability evaluation of the reservoir islands levees indicates that as much as 2 feet of downslope deformation on the reservoir side and 4 feet of deformation on the slough side could be experienced during a probable earthquake in the region.
- As indicated by DW, it is planned, as a part of final design, to implement extensive and detailed subsurface exploration programs along the reservoir island levees, followed by stability evaluations and site-specific detailed design and construction to provide adequate levee stability. These steps will be essential to achieve safety and effectiveness of the proposed levee system.

Overall Findings

Taking a broader view, we consider the overall findings of this reevaluation of geotechnical issues of the proposed Delta Wetlands Project to be as follows:

- The seepage mitigation design proposed by DW appears appropriate and has the potential to be effective, provided that
  - the interceptor well system is appropriately designed, constructed and operated,
  - the monitoring system consisting of seepage monitoring wells and background wells is appropriately designed, constructed and operated, and
  - the significance criteria are rigorously applied and continually updated based on experience.
- The levee strengthening conceptually proposed by DW appears appropriate, except that measures need to be developed to improve the stability of the raised levees toward the slough.
- Because conditions around the islands' perimeter vary, it will be essential that a "mile-by-mile" geotechnical exploration and, based on it, a detailed final design, be implemented. The exploration should consist of borings and soundings spaced closely enough that adverse

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conditions extending over some distance would be identified. Appropriate detailed geotechnical laboratory tests, in particular grain size, permeability and strength tests, should be made on recovered samples. Final design of seepage control and monitoring, and levee strengthening, should consider the specific conditions identified on a site-specific basis.

- Construction of the improvements will require detailed geotechnical construction oversight, construction quality control and quality assurance, and documentation of as-built features, to maximize the chances that unexpected conditions are identified and accommodated, that construction will be implemented to satisfy the intent of the design, and that construction is documented.
- In particular, the design, construction, and operation of extraction wells will be critical to maximize the reliability of the seepage control system. It will also minimize the possibility of flushing fine particles out of the levee foundation, which could over time lead to weakened levee foundations and potential settlement and stability problems.
- It is recognized that pumping from the crest of the reservoir levee to mitigate seepage effects across the slough in the adjacent island is not the most effective way to achieve the seepage mitigation. It has been selected because of ownership and access issues. Other measures to achieve the seepage mitigation could be developed. In particular, pumping from the adjacent islands' levee across the slough from the reservoir islands would be hydraulically more efficient, and would likely require fewer wells and lower pumping volume. Passive or active relief wells or trenches on the adjacent islands would also be effective. A continuous cutoff around the reservoir islands would also be effective, but would likely be cost prohibitive.

1.1 DESCRIPTION OF DELTA WETLANDS PROJECT

The Delta Wetlands (DW) Project's purpose is to provide water storage in the Sacramento/San Joaquin River Delta. The project plans to store excess runoff water during heavy winter and spring runoff on two Delta islands and release the water later in the year for beneficial use. The planned reservoir islands are Bacon Island and Webb Tract, shown in Figure 1.1.1. In addition, two islands are planned to be converted to wetlands for environmental mitigation; these islands are Bouldin Island and most of Holland Tract, also shown in Figure 1.1.1. The project is fully described in the Draft EIR/EIS prepared by Jones & Stokes Associates (1995).

Conceptual plans for the conversion of the reservoir islands from agricultural use to water storage use have been developed by DW and are also described in the Draft EIR/EIS. A brief summary of the proposed concept is provided below.

The levees around the reservoir islands will be raised and strengthened, such that the islands could store water up to a maximum elevation of +6 feet (all elevations are related to NGVD). Erosion protection on the levees will be raised on the channel side and will also be provided on the reservoir side. Siphons with supplemental pumps will be installed to fill the reservoir islands during periods of surplus flows through the Delta. Pumps will move the stored water back into the Delta waterways when it is needed by the users. The reservoir islands could be completely emptied by pumping.

Sandy fill for levee raising and strengthening will be obtained from the interior of the reservoir islands far from the levees. Surficial peat will need to be excavated to reach the suitable sandy fill soil. Disposition of the excavated peat overburden is at the discretion of DW. It could be backfilled into the excavation after sand removal, but this is not necessary for seepage control if the excavations are at least 800 feet from the levee (as shown later in this report).

A system of a large number of extraction wells installed on the levees of the reservoir islands has been proposed by DW to protect the islands adjacent to the reservoir islands from the anticipated effects of seepage from the reservoir islands. Such seepage effects are expected because of a deep sand aquifer that underlies the reservoir and the adjacent islands as well as the channel or slough separating them. This layer facilitates movement of water from the reservoir islands (with a higher water table) to adjacent islands. To control the amount and duration of pumping from these wells to such an extent that the effect on the adjacent islands is small or imperceptible, DW has proposed a complex monitoring system. The system would include both seepage monitoring wells immediately across the channel from the reservoir islands and background wells to establish water-level changes that could occur unrelated to water storage on the reservoir islands. DW has also proposed significance criteria that will provide the method by which monitoring well data are used to control pumping of the extraction wells and to provide threshold levels that would trigger emergency response.

1.2 REASONS FOR SUPPLEMENTAL EVALUATION

The primary reason for this supplemental geotechnical evaluation of the Delta Wetlands Project is to address concerns expressed by the State Water Resources Control Board (SWRCB) in a letter dated November 25, 1998. The SWRCB's letter is included herewith in Appendix C. The letter raised a number of questions related to geotechnical issues included in the Draft EIR/EIS

(Jones & Stokes, 1995). Specific issues raised in Section III of Attachment A to the SWRCB letter included several aspects of seepage, seepage control by interceptor wells, and seepage monitoring; and levee stability aspects. A decision was then made that a supplemental EIR/EIS (referred to as Revised EIR/EIS or REIR/EIS) for the project should address these issues and provide more detailed evaluation of the geotechnical issues of the project. A decision was then made by Jones & Stokes Associates to engage URS Greiner Woodward Clyde (URSGWC) to provide the supplemental geotechnical evaluations and prepare a supplemental geotechnical report. The present report is the result of this work.

1.3 DEVELOPMENT OF SCOPE OF EVALUATIONS

In response to a request by Jones & Stokes Associates, URSGWC prepared a scope of work dated June 25, 1999 to address the specific geotechnical issues raised in SWRCB's letter and attachment. The scope included a relatively brief review of prior work, since additional more detailed reviews would be conducted under the specific work tasks. In response to comments by Delta Wetlands stating that they had already implemented a portion of the proposed evaluation, URSGWC developed a revised scope dated July 6, 1999 that included a two-phase study. Phase 1 involved a more detailed review of prior work by Delta Wetlands, including review and responses to a detailed letter by Delta Wetlands dated August 3, 1999, where they pointed out issues they felt they had adequately covered in previous studies. Phase Two involved the basic geotechnical evaluation scope, incorporating any changes to the remainder of the proposed work. Subsequently Phase 1 was authorized, and our report on this phase, dated September 13, 1999, contained a revised scope of work for the geotechnical evaluations and responses to Delta Wetlands' August 3 letter. This revised scope was subsequently authorized, and has been implemented. Copies of the original scope, comments from DW, and a revised scope are included in Appendix D.

1.4 OUTLINE OF REPORT

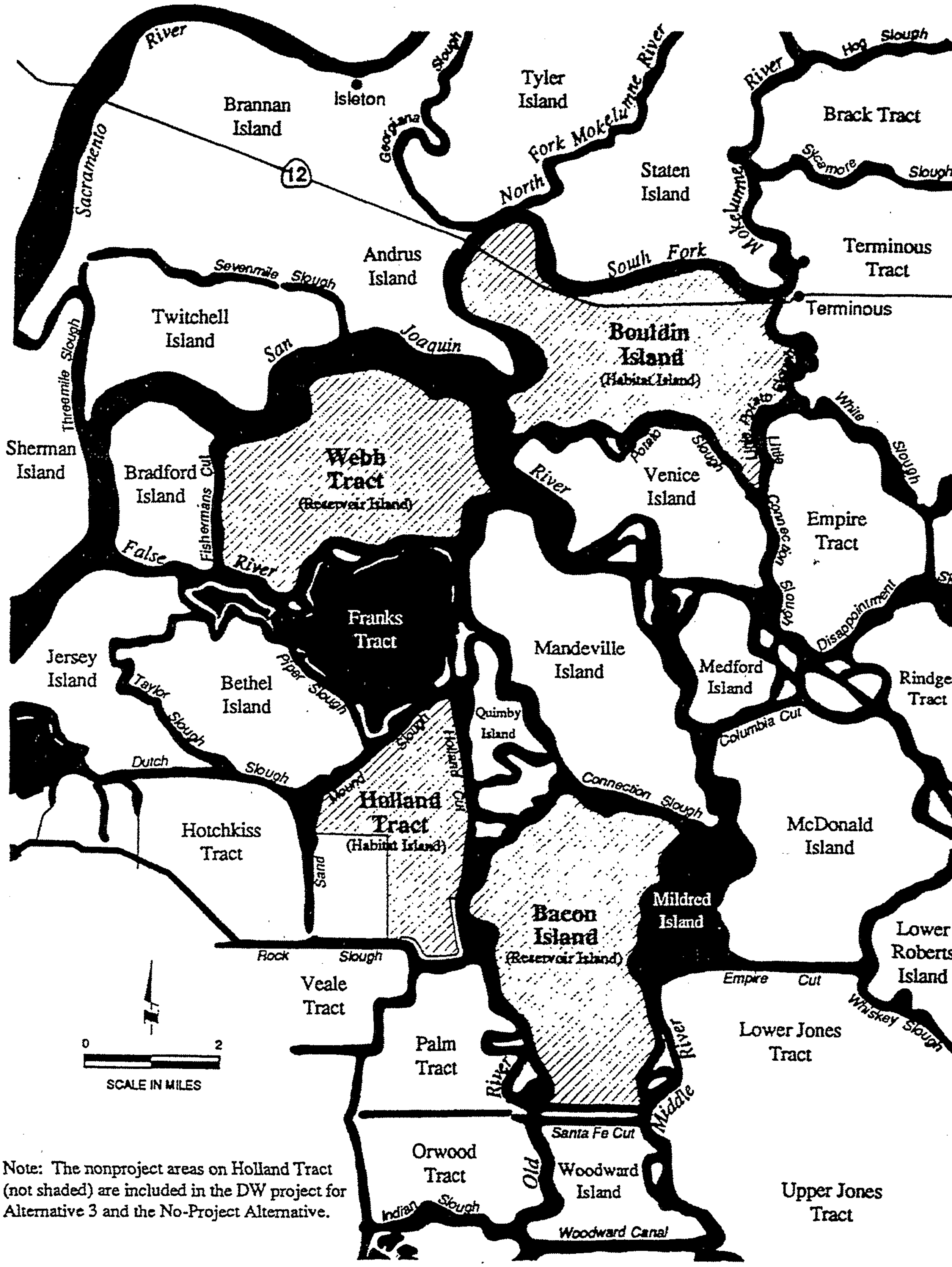
This report is organized essentially according to the proposed revised scope of work. Section 2 addresses all seepage issues, and includes in turn the objectives, review of prior work, seepage analyses for conditions without and with interceptor wells and their results, and review of the proposed monitoring system including proposed "significance standards." Additional items addressed in Section 2 include maintenance and reliability of interceptor wells, potential water diversions, and potential settlements due to operation of the reservoir islands.

Section 3 addresses levee stability issues. Included in turn are objectives, review of prior work, static stability analyses, seismic stability/deformation analyses, and seismic and geologic hazards. Further included are estimates of levee settlements and their effects on stability, slope erosion/scour, review of borrow requirements, and assessment of potential effects of interceptor wells on stability.

Section 4 summarizes the key findings from our evaluation of seepage and stability issues

Section 5 notes limitations of our evaluations.

Section 6 contains references.



Note: The nonproject areas on Holland Tract (not shaded) are included in the DW project for Alternative 3 and the No-Project Alternative.

Source: Adapted from California Department of Water Resources 1993.

Source: from Jones and Stokes, 1995

Project No. 41-07099030.00	Delta Wetlands	DELTA WETLANDS PROJECT ISLANDS	Figure 1.1.1
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2.1 SEEPAGE ANALYSIS OBJECTIVES

Active interceptor well systems have been proposed by Delta Wetlands (DW) to mitigate potentially detrimental seepage impacts on neighboring islands as a result of filling the proposed reservoirs at Webb Tract and Bacon Island. In response to the SWRCB concerns about the feasibility, adequacy and effectiveness of the proposed interceptor well system, we have performed independent seepage analyses to evaluate the proposed system. Seepage analyses have been conducted for the conditions anticipated at four different locations along the reservoir islands. Included in the evaluation of the interceptor well system were:

- Review of previous seepage studies, including review of the subsurface conditions and material properties.
- Generation of two-dimensional finite element models for four locations to simulate various seepage conditions.
- Evaluation of the effects of proposed reservoirs on existing seepage conditions and the required performance of the interceptor well system.
- Evaluation of the effects of proposed borrow area locations on seepage conditions and the performance of the interceptor well system.
- Completion of sensitivity analysis, in which critical parameters used in the seepage models are varied.

In addition, an evaluation of the effectiveness of the proposed monitoring system and procedures has been completed. The monitoring system for groundwater seepage, developed by HLA, is to provide a standard of performance against which project related seepage can be determined. Using the results of the seepage modeling and reviews of the proposed monitoring system, the adequacy and effectiveness of the proposed procedures was assessed, including the criteria (termed "significance standards") developed to determine whether observed seepage conditions merit mitigating action.

An evaluation of potential water diversions was made using the seepage models created for the two islands. This evaluation was required to address SWRCB's concern that, during certain water level conditions in the storage islands and constraints on permissible DW operations, pumping from the interceptor well system may constitute water diversions from adjacent channels into the storage islands. The seepage models have been used to estimate such diversions.

Finally, settlements that may be caused by reservoir filling and pumping were estimated. Rapid reservoir filling, drawdown, and groundwater pumping may induce additional soil stresses that may lead to additional settlements of levees and island interiors.

We note here that all elevations in this report refers to the National Geodetic Vertical Datum (NGVD).

2.2 REVIEW OF PREVIOUS SEEPAGE EVALUATIONS

Harding Lawson Associates (HLA) and then Hultgren Geotechnical Engineers (HGE) have performed geotechnical studies on the proposed project since 1988 on behalf of the project owner DW. The studies included collecting data from site explorations and performing analyses to address geotechnical engineering concerns including settlement, erosion, seepage, stability and seismic hazards. A number of reports were prepared during the previous studies, and the Reference section lists these reports. In this section a review of the previous studies related to the seepage issues of Webb Tract and Bacon Island is presented.

As a part of a preliminary geotechnical investigation in 1989 (HLA 1989), the following subsurface exploration was performed:

Webb Tract	Twenty-six Cone Penetration Tests (CPTs) for subsurface characterization, and seven borings and four monitoring wells around the island perimeter for subsurface sampling and characterization of soils and groundwater levels.
Bacon Island	Twenty-one CPTs, eight borings and four monitoring wells around the island perimeter.

Figure 2.2.1 shows locations of some of the CPTs and borings in Webb Tract, (those in the vicinity of our analysis sections) and Figure 2.2.2 shows similar information for Bacon Island. Soil borings and CPTs were located on the levees and in the interior of the islands to characterize the site stratigraphy. Soil samples from the borings were selected by HLA (based on stratigraphy and need for information) for laboratory testing, including moisture content, dry density, shear strength, compressibility, grain size, specific gravity and permeability.

2.2.1 Typical Stratigraphy of Interior Island

From the investigations, it was found that the general stratigraphy of the interior of Webb Tract and Bacon Island was similar. In general, the interior stratigraphy consisted of a surfacial soft, organic fibrous peat layer underlain by a silty sand aquifer, below which lies stiff silty clay. These units are laterally continuous and relatively constant in thickness. In some areas, deeper sand aquifers are present below these units. Soil borings for the groundwater monitoring wells indicate that a similar stratigraphy exists on adjacent islands. The thicknesses of the peat and sand layers vary from one part of the islands to another. The sand layer is exposed in some portions of Webb Tract.

2.2.2 Typical Levee Condition

Typical levee conditions of the islands consist of a layer of fill about 10 feet thick consisting mostly of sand with some peat and clay. The fill is typically underlain by peat and soft clay that in turn is underlain by a sand aquifer and deeper silty clay layer.

Because the levee was originally constructed at about sea level and levee settlement and raising have occurred periodically since initial construction, it is likely that the upper portion of the peat and soft clay is also fill. It was not possible during the explorations to differentiate this soil from the undisturbed native peat or clay.

2.2.3 Soil Permeability Used in Prior Studies

Table 2.2.1 presents vertical and horizontal permeabilities of the existing soil layers. The vertical permeability was measured by the laboratory tests, and the horizontal permeability was estimated by typical anisotropy ratios for similar deposits. The values presented in Table 2.2.1 were used in the HLA (1989) report to develop a preliminary computer-based seepage model. Additional data collection (pump tests and laboratory permeability tests) was recommended as part of final design of the project to verify that the permeability values used in the analysis are reasonable.

Pump Test Results: After the 1989 preliminary geotechnical investigations, DW and HLA performed two constant rate pump tests, one on Holland Tract and the other on McDonald Island (HLA 1991b).

The pump test on Holland Tract was conducted from April 24 through 26, 1989. A pumping well and four observation piezometers (two deep, fully penetrating the sand aquifer, and two shallow piezometers in peat layers extending to 8.5 feet) were installed. During the testing, a constant discharge rate of 30 gallon per minute (gpm) was maintained. Based on the analysis of the data, permeability values were estimated at 15.3 feet/day (5.4×10^{-3} cm/sec) at one deep observation well (20 feet from the pumping well) and 18.2 feet/day (6.4×10^{-3} cm/sec) at the other deep observation well (30 feet from the pumping well).

The pump test on McDonald Island was performed from August 15 through 16, 1989 as a part of the Phase I drawdown demonstration. Brief details of the Phase I as well as Phase II (or final phase) drawdown demonstration are presented in the next section. A constant pump rate of 215 gpm was maintained during the test. The estimated permeability value for this test was 390 feet/day (1.4×10^{-1} cm/sec) . Because the tidal fluctuations had an influence on the drawdown data, HLA used a distance vs. drawdown method (instead of time vs. drawdown) for the estimation of this value.

The average permeability value of 16.8 feet/day (5.9×10^{-3} cm/sec) for the Holland Tract test site corresponded to a very fine to fine grained, poorly graded sand with silty sand. The permeability value for the McDonald Island represented medium grained silty sands with gravel. HLA indicated that this was the coarsest material encountered as part of the investigations.

2.2.4 Field Drawdown Demonstration Studies

A drawdown demonstration was performed by DW and HLA on a quarter-mile long levee on McDonald Island. The purpose of the demonstration was to evaluate whether the hydraulic head within the sand aquifer could be lowered by pumping from interceptor wells and by using gravity flow relief wells. (Pumped interceptor wells use submersible pumps to draw water from the wells, whereas relief wells simply use passive flows from the wells into low lying ditches to relieve some of the water pressure in the sand aquifer.) If effective, systems using pumped interceptor wells or gravity flow wells could be used to control the seepage resulting from the operation of the DW reservoir islands. During Phase I of the demonstration, the pumped interceptor well system was studied, and in Phase II the gravity flow relief well system was studied.

During the Phase I demonstration (HLA 1990a), an interceptor well system consisting of 15 wells was tested. The wells were located on the levee with an approximate spacing of 125 feet. The 6-inch diameter wells were screened within the entire sand aquifer thickness. All wells were connected by a 12-inch diameter header pipe, which was connected to a suction pump with a capacity of approximately 1,500 to 1,800 gpm. Average flow rates for individual wells ranged from 75 to 90 gpm and the total system flow was between 1,100 to 1,300 gpm. During the operation of the interceptor well system, the water elevation in the sand aquifer became flat with an average elevation of about -16 feet (the elevation before pumping was -13.3 feet). It was concluded by HLA that the pumped interceptor well system was effective in controlling the seepage.

Phase II (HLA 1990c) used the same wells, modified to a passive flow relief well system by connecting the tops of the wells to drainage ditches dug three to four feet below the ground surface. In the passive well system, groundwater flows from wells into seepage ditches due to the artesian pressure in the sand aquifer. The total average discharge from the 14 wells was approximately 600 gpm, and the average discharge per well was approximately 44 gpm. Water levels in the sand aquifer were lowered to -15 feet elevation beneath the island interior. It was concluded by HLA that the gravity well system also was effective in controlling the seepage, but achieved somewhat less water level drawdown.

HLA reported that settlement rates increased slightly during the drawdown test, and explained that these increased rates were due to the fill material that had been recently placed and due to the lowering of the water table around the wells. They noted, however, seepage control measures installed by Delta Wetlands would maintain water levels within historic ranges, and that no increased ground loading and corresponding settlement should result.

Following the McDonald Island drawdown tests, there was some question regarding the long term effectiveness of the interceptor well system. Specifically, during the rebuttal testimony of Mr. Edwin Hultgren (July 31, 1997), a question was raised as to why the fields of Mr. Alfred Zuckerman on McDonald Island again became saturated and unfarmable after the drawdown test was completed and only the relief wells remained operating. The response was that the relief wells had become less efficient in drawing down the water table with time because they had become clogged with silt. Mr. Hultgren explained that the wells were not designed and built for long-term operation, and they were not maintained once the test program was completed.

2.2.5 Background Groundwater Monitoring

A groundwater monitoring program was established to provide regional groundwater elevations in the islands before the construction of the Delta Wetlands project (HLA 1990b, 1991a, 1992f, 19995c, 1995d). Data collected before project construction would provide baseline information on the existing condition. The baseline information was intended to be used for the evaluation of seepage due to the project. The groundwater monitoring program consisted of 32 monitoring wells located in 17 Delta Wetlands and adjoining islands. Figure 2.2.3 shows the location of the monitoring wells. Data collection began in February 1989, and continues today. Water levels are measured manually at a weekly frequency. The monitoring data were presented in a number of reports (HLA 1990b, 1991a, 1992f). From the data, it was concluded that the groundwater levels varied with the tidal fluctuations in nearby sloughs and rivers. It was also found by HLA that the groundwater variations over a year could be fitted either with a straight line or with a simple harmonic (sine function) curve (HLA 1995c, 1995d).

2.2.6 Interceptor Well Modeling Results from HLA

HLA performed groundwater computer modeling to simulate the control of seepage into neighboring islands using various interceptor well systems located on the DW island levees (HLA 1991b). The purpose of the simulation was to establish parametric relationships to develop the basis of a conceptual design of an interceptor well system. Simulation was performed using a two-dimensional, steady state flow, finite difference program called FLOWPATH. The modeling considered the following range of parameters:

- Two types of aquifer systems (one confined aquifer, and one unconfined aquifer starting 100 feet from the perimeter levee),
- Three transmissivities (200, 3,500 and 20,000 ft^2/day),
- Three interceptor well spacings (80, 160, and 320 feet), and
- Two borrow pit excavations (borrow pits were simulated in confined aquifers; borrow pits were assumed to be excavated into the aquifer at 2,000 and 400 feet from the levee; each borrow pit was 500 feet wide).

Several FLOWPATH runs were performed for various combination of the above parameters. The results provided a range of pump rates corresponding to well spacings and aquifer parameters.

Based on the modeling it was concluded by HLA that an interceptor well system installed on the perimeter of the reservoir islands was a viable solution to control seepage. Furthermore, a possible interceptor well system location was presented. The interceptor wells were estimated to cost $120,000 (1991 dollar value) per mile of levee. This estimate was based on a well spacing of 160 feet, and 50-foot deep wells equipped to pump at 70 gpm.

2.2.7 Main Findings and Conclusions from Previous Studies

The following text summarizes the conclusions drawn by HLA and DW based on their studies:

- There is a possibility of increased seepage into the neighboring islands due to the storage of water in the reservoir islands, if no mitigation is implemented.
- The islands' interior stratigraphy generally consisted of peat underlain by a silty sand aquifer, below which lies stiff clay. These units are laterally continuous, but the thicknesses of the peat and sand layers were observed to vary somewhat from one part of the islands to another.
- Vertical soil permeability values were measured in the laboratory, and drawdown pump tests were performed to determine soil permeability values. However, the majority of the horizontal permeability values were estimated based on the gradation of the soil.
- The groundwater level beneath the levees is generally near sea level. Beneath the island interiors, the water head in the sand aquifer is generally five feet below the ground surface.

In some locations, where artesian conditions exist in the confined aquifer, the head is as much as five feet above the ground.

- A program of background regional groundwater level measurements was started in February 1989, and this monitoring program still continues today. Thirty-two monitoring wells are used in this program. Based on the data collected, it was found that the groundwater levels varied with tidal fluctuations in nearby sloughs and rivers and also with the seasons. It was also found that the groundwater variations over a year could be fitted either with a straight line or with a simple harmonic (sine function) curve.
- It was concluded by HLA that the drawdown test on McDonald Island showed that the interceptor well system would be effective in controlling seepage. Regarding the loss of effectiveness of the relief wells with time, HLA explained that the wells were not designed and built for long-term operation, and they were not maintained once the test program was completed. Although minor settlement occurred during the drawdown test, HLA does not anticipate any ground settlement associated with the proposed interceptor well system proposed for the Delta Wetland project.
- Interceptor well modeling showed that an interceptor well system installed on the perimeter of the reservoir islands could be a viable system to control the seepage into the neighboring islands.

URSGWC generally agrees with these findings, but offer the following additional comments:

In our opinion, the McDonald Island drawdown test provides valuable information on the effort required to draw down the groundwater table to acceptable levels on neighboring islands. However, the actual pumping conditions that the proposed interceptor well system will experience will be more severe than those seen at McDonald Island could. On the reservoir island levees, the interceptor wells will be working against a higher head (being so close to the reservoir) and will have to pump at a higher rate to intercept the reservoir-induced seepage and lower the groundwater table on the neighboring island. Also, even though the sand aquifer underlying McDonald Island is similar to that underlying Webb Tract and Bacon Island, the sand aquifer at the test location on McDonald Island was overlain by a confining layer of silt. This overlying layer, which effectively reduces the groundwater seepage rates toward the ground surface, is not present everywhere on Webb Tract and Bacon Island. Subsurface investigations indicate that on most of Webb Tract and Bacon Island, only a thin layer of peat overlies the sand aquifer. Without the silt layer, the interceptor well system would have to pump at a higher rate to effectively lower the groundwater table. In addition, the proposed interceptor well system will be located on the reservoir island levee, not inside the levees of the neighboring island.

- The drawdown test at McDonald also provides valuable information on the response of the sand aquifer to pumping. The sand aquifer beneath McDonald Island appears to be hydraulically similar to that under Webb Tract and Bacon Island, based on gradation tests performed on samples taken in the aquifers. The hydraulic conductivity of the sand aquifer is controlled by the proportion of fine materials present, as shown by the relationship given by Cedergren (1989)

$$k = C \times D_{10}^2$$

Where:

k = hydraulic conductivity (cm/sec)

C = constant (approximately 100)

D_{10} = diameter (cm) of soil particle below which 10% of the sample particles are smaller

This approximate relationship shows that the D_{10} values control the hydraulic conductivity of the material. From the sand samples taken at McDonald Island, the average D_{10} value is approximately 0.007 cm, and the corresponding calculated hydraulic conductivity is about $5x10^{-3}$ cm/sec. From the range of typical gradations given for aquifer samples taken at Webb Tract and Bacon Island (HLA 1989, Plate B-1), the average D_{10} values are approximately 0.005 to 0.006 cm, and the corresponding calculated hydraulic conductivities are about $2.5x10^{-3}$ to $3.5x10^{-3}$ cm/sec. This indicates that the sand under McDonald is slightly more pervious than that under Webb Tract and Bacon Island.

- In our opinion, the drawdown tests at McDonald Island show that potential migration of fine materials from the sand aquifer to the pumping system is of concern at the interceptor wells proposed for Webb Tract and Bacon Island, and the wells will have to be carefully designed and constructed to maintain their effectiveness and minimize migration of fines from the aquifer into the well. Regular maintenance and redevelopment of the wells will be required to restore pumping efficiency when required. Monitoring of ground surface elevations near the interceptor wells should be performed to observe any minor ground subsidence that may occur due to potential loss of fine materials from the underlying sand aquifer. A record of required well maintenance should also be kept to identify any wells that might have silt losses.

2.3 SEEPAGE ANALYSES

2.3.1 Seepage Analysis Approach

Previous Models. Previous seepage models used by DW to analyze the interceptor well system used plan view modeling techniques to estimate seepage conditions within the sand aquifer only. Those plan view models considered seepage conditions within the sand aquifer (considering the aquifer as being either confined or unconfined) over a large area, extending 3000 feet on either side of the interceptor well system. The boundary conditions for the plan view models were established a large distance (over 2000 feet) from the interceptor wells, where a constant head boundary was used to simulate the reservoir and adjacent island background conditions.

The limitations of this modeling approach include the fact that the plan view model only considers the seepage conditions within the sand aquifer. While a significant portion of the seepage will occur within the aquifer, the effects of the other elements of the subsurface stratigraphy are not seen. In addition, the plan view model does not consider the influence of surface water infiltration from the proposed reservoirs or the existing sloughs. Neglecting the effects of surface water infiltration will limit the plan view model's ability to simulate localized seepage conditions near the interceptor well system.

Current Seepage Analysis Approach. To evaluate the effectiveness of the active interceptor well system proposed for Webb Tract and Bacon Island, two-dimensional cross-sectional finite element models were used to simulate seepage conditions and estimate the required pumping effort at the interceptor well system. The cross-sectional modeling approach was chosen as it considers all major elements of the subsurface stratigraphy at each section. The models were built to simulate seepage conditions along sections taken perpendicular to the levees and sloughs, and were developed to model average conditions in close proximity to the interceptor well system. The potentially significant effects of surface water infiltration from both the slough and proposed reservoir can be modeled using this method.

The drawdown condition along the line of interceptor wells that is induced by pumping is expected to vary significantly along the levee. Figure 2.3.1 shows an example of a plan view model for a 50-foot thick confined sand aquifer with boundary conditions similar to those anticipated near the interceptor well systems on Webb Tract and Bacon Island. As shown on Figure 2.3.1, the total head conditions along the line of wells spaced at 160 feet varies considerably, with the maximum amount of drawdown occurring at the pumping wells.

In order to represent this drawdown effect in the cross-sectional models, an average total head along the interceptor well line (as shown on Figure 2.3.1) was used to model average drawdown conditions along the levee. All of the cross-sectional models developed for this seepage analysis therefore generate average seepage conditions across the section of levee considered. Average pump rates along the levee estimated by the cross-sectional models (presented in gallons per minute (gpm) per foot of levee) can be converted to actual pump rates at a single well by multiplying the average pump rate by the well spacing used.

The cross sectional models developed for the seepage analysis were used to estimate parameters that were considered critical for the evaluation of the influence of the proposed reservoirs and the interceptor well system. Specific parameters include:

- The average total head (in feet) in the sand aquifer near the levee centerline (reservoir island).
- The total seepage flow through a vertical section, termed the seepage flux (in gpm per foot of levee), near the levee centerline.
- The average total head (in feet) in the sand aquifer at the far (adjacent island) levee centerline.
- The flux quantity (in gpm per foot of levee) at the far levee centerline.
- The water table level at the far toe of the far levee (near the ditch).

The water table level at the far toe was considered to be an important indicator of impacts detrimental to adjacent islands, as a significant rise in the ground water table may impact agricultural production rates.

A description of the transverse sections modeled for Webb Tract and Bacon Island is presented in Section 2.3.2. Included in the description is the subsurface stratigraphy at each location, the hydraulic properties of each material within the model, the model's boundary conditions and the seepage conditions considered.

Computer Model. The computer program SEEP/W (Geo-Slope International Ltd., 1994) was used to estimate seepage conditions through transverse sections of the existing levees at Webb

Tract and Bacon Island. SEEP/W uses a two-dimensional finite element method to model seepage conditions and assumes that flow through both saturated and unsaturated media follows Darcy's Law. (Finite-element modeling is generally considered to be similar to or more effective than the finite-difference modeling used by DW.) The seepage analyses were conducted considering steady-state conditions.

Using the SEEP/W mesh generation program, finite element meshes were generated to model the multiple seepage conditions considered for the levees on Webb Tract and Bacon Island. The element material types are represented in the models as different colors, as shown on Figure 2.3.2. Fixed boundary conditions were used to model constant reservoir and slough heads, heads within pumping wells and far-field groundwater levels. Other portions of the levee and ground surfaces on the islands were modeled using an unrestricted, free-flowing boundary condition; that is, a boundary condition that is determined at each node by SEEP/W during the analysis of flow conditions. The bottoms of the cross sections were modeled as no-flow boundaries.

The SEEP/W analysis program was used to evaluate the steady-state phreatic surface location, the head distribution throughout the model and flow quantities at particular locations. The SEEP/W contouring program was used to generate head distribution diagrams. Phreatic surfaces, total head contours (in feet of water) and flux quantities (in gallons per minute per foot width of levee) are presented on each of the figures presenting the analysis results for each section. The flux quantities represent the flow quantity across the length of a particular flux section, which is symbolized as a blue arrow on the figure.

2.3.2 Analysis Sections and Boundary Conditions

Analysis Sections. Four sections were considered for the seepage analysis, two at Webb Tract and two at Bacon Island. The locations of the Webb Tract sections, at Stations 260+00 and 630+00, are shown on Figure 2.2.1. The locations of the Bacon Island sections, at Stations 220+00 and 665+00, are shown on Figure 2.2.2.

For each island, one section was chosen to model more critical seepage conditions (Webb Tract Station 630+00 and Bacon Island Station 220+00), considering both the subsurface conditions and the geographic conditions relative to adjacent islands. More critical seepage conditions are expected to occur at locations where the slough is narrower, where the sand aquifer is thicker, or where less pervious materials that overlie the sand aquifer (such as peat or channel silt) may be thinner. The other two sections were chosen at typical but less critical locations where subsurface conditions were available to consider the effects of varying conditions and to provide a range of analysis results, including flow rates, phreatic surface locations and required pump rates. It should be noted that these are not the least critical locations on the islands (which occur at locations like those adjacent to the San Joaquin River at Webb Tract where there is no nearby adjacent island), but instead are less critical locations chosen, after review of the range of levee and subsurface conditions, to model varying surface and subsurface effects on the interceptor well system.

The subsurface conditions at Webb Tract Stations 260+00 and 630+00 and the approximate thickness of each layer are presented on Table 2.3.1. This stratigraphy is based on field investigations performed previously by others (see Section 2.2). Typical subsurface conditions at the levees along Webb Tract include levee fill material (clay with peat and sand) underlain by native peat. An approximately 50-foot thick layer of sand underlies the peat layer. The sand aquifer is underlain by a clay layer of relatively low hydraulic conductivity. Also included in the model is a channel silt deposit, with an estimated thickness of three feet (see next paragraph), and the proposed new fill material for the land-side portion of the levee. The simplified subsurface stratigraphy at Stations 260+00 and 630+00 is shown on Figures 2.3.4 and 2.3.2, respectively.

We could not locate any direct data on thickness, permeability and continuity of the channel silt. The best "proof" of the reasonableness of the assumptions made is the analysis of the present conditions (without project), which looks reasonable with the channel silt as assumed. The sensitivity analysis using higher permeability in the channel silt indicated that the neighboring islands would experience serious seepage problems, which is not the case. It was also decided at a project meeting that dredging of the channel silt would not be considered in the evaluation of the Delta Wetlands Project, because the effects of such dredging would have to be addressed and accommodated by whoever planned to dredge the channels.

The subsurface conditions at Bacon Island Stations 220+00 and 665+00 and the approximate thickness of each layer are also presented on Table 2.3.1. Typical subsurface conditions at Bacon Island Station 220+00 include levee fill material (clay with peat and sand) underlain by native peat. An approximately 20-foot thick layer of sand underlies the peat layer. The sand aquifer is underlain by a clay layer of relatively low hydraulic conductivity. Typical subsurface conditions at Bacon Island Station 665+00 include levee fill material (clay with peat and sand) underlain by native peat and an upper layer of relatively low hydraulic conductivity clay. An approximately 20-foot thick layer of sand underlies the upper clay layer. The sand aquifer is underlain by a lower clay of low hydraulic conductivity. Also included in both models is a channel silt deposit, with an estimated thickness of about three feet, and the proposed new fill material for the land side portion of the levee. The simplified subsurface stratigraphy at Stations 220+00 and 665+00 is shown on Figures 2.3.6 and 2.3.8, respectively.

Analysis Conditions. For each section considered at Webb Tract and Bacon Island, three seepage conditions were evaluated: (1) existing conditions, (2) with-project, full reservoir with no pumping at the interceptor wells, and (3) with-project, full reservoir with required pumping at the interceptor wells. Existing conditions were first analyzed to calibrate the model against field observations and to verify that the boundary conditions and material properties were appropriate. Full reservoir conditions with no pumping were analyzed as an intermediate condition to estimate the effects of a loss of pumping capacity on the neighboring islands. Full reservoir conditions with pumping at the interceptor well system were analyzed to evaluate the effects of the proposed interceptor well system. The minimum pump rate (in gallons per minute per foot of levee) required to retain pre-reservoir seepage conditions at the far levee was estimated.

In addition to the three cases described above, additional analyses were performed to evaluate the sensitivity of the results to variations in material properties and to the location of proposed borrow pits. Sensitivity analyses were conducted by varying the hydraulic conductivities of the channel silt and the aquifer sand, and by varying the thickness of the peat layer on the land side of the levees. The proposed borrow pits, which were assumed to be 500 feet wide and extend to the sand aquifer, were modeled at locations of 400 and 1000 feet away from the levee, and were assumed to allow direct inflow of water into the aquifer. These sensitivity analyses were conducted only for Webb Tract at Station 630+00.

Boundary Conditions. The primary boundary conditions affecting the seepage models include the constant head boundaries imposed by presence of the slough, the full reservoir, and the groundwater conditions within the adjacent island. The slough was modeled as having a constant elevation head of +1 feet (using the NGVD elevation datum). The slough level at the islands will vary up to about three feet between daily high and low tides; however, the average daily value of +1 feet was considered representative for the model. The average daily value was considered appropriate because tidal fluctuations at the surface are not expected to significantly influence conditions within the confined sand aquifer at any point in time. For the full reservoir condition, a constant elevation head of +6 feet was used, based on our understanding of expected reservoir operation levels.

The far-field boundary condition at the neighboring island under existing conditions was estimated through a calibration procedure. The model meshes were constructed so that the far-field boundary conditions occurred at a significant distance from the levees (i.e., about 600 feet from the levee at Webb Tract Station 630+00) so that the boundary reflected background groundwater conditions. The far-field head was iteratively varied until the phreatic surface estimated by the model matched the observed groundwater levels about 2 to 3 feet below the surface observed in piezometers and ditches on the islands. Once the far-field boundary condition was established on the adjacent island, it was held constant for the other two full reservoir conditions.

For the full reservoir condition with pumping at the interceptor wells, a constant head boundary was also placed through the sand aquifer at the location of the well line. This boundary condition was used to represent the average total head along the well line during pumping, and was varied to determine the required pump rate. The actual pump rate (gpm per foot of levee) was determined by estimating the flow rates at the well under the pumping head conditions.

2.3.3 Hydraulic Conductivities

As mentioned in Section 2.2, several analyses have been previously performed by others to estimate the subsurface materials' hydraulic conductivities at Webb Tract and Bacon Island. These analyses have included laboratory tests, field pump tests and estimates made using material gradation characteristics. Considering the results of these previous studies, we have used the hydraulic conductivities shown on Table 2.3.1. As shown, the fill material, peat, and sand were all modeled with an anisotropy (the ratio of horizontal to vertical hydraulic conductivity) of 10. Previous studies have shown an anisotropy of up to 100 for peat; however, a more conservative factor of 10 (using a higher vertical conductivity of 1×10^{-4} cm/s) was used for these analyses.

Variations in the hydraulic conductivities of the channel bottom silt and aquifer sand were made for the sensitivity analyses, as these two materials were expected to have a large influence on the overall seepage conditions at the levees. The channel silt controls the infiltration rate of water seeping from the slough, and the aquifer sand permeability may have the greatest influence on overall subsurface flow rates beneath the levees. For the sensitivity analysis, the hydraulic conductivities of the channel silt and aquifer sand were each separately increased by a factor of 5. These values were chosen to reflect the variations of field conductivities considered reasonable for the channel silt, and to consider the estimate of the sand's hydraulic conductivity derived from the McDonald Island pump test (where 5.4×10^{-3} to 6.2×10^{-3} cm/sec was estimated, see also Section 2.2.7). In addition, a sensitivity analysis was conducted by halving the peat layer thickness over the island from six feet to three feet. The island peat thickness and permeability control the infiltration rate of water seeping from the reservoirs, provided (as shown later) that the borrow pits are located at least 800 feet away from the levee.

2.3.4 Analyses Results

Figures 2.3.2 through 2.3.9 present the seepage analyses results for Webb Tract Stations 260+00 and 630+00, and Bacon Island Stations 220+00 and 665+00. Each set of two figures presents, for one cross section, (1) the cross-section stratigraphy, model mesh and hydraulic conductivities, (2) the existing seepage conditions, (3) the seepage condition corresponding to a full reservoir with no pumping at the interceptor wells, and (4) the seepage condition corresponding to a full reservoir with required pumping at the interceptor wells. On all figures, total head contours (in feet) are drawn across the entire section. (Note that the program SEEP/W automatically draws total head contour lines above the phreatic surface as well as below, however it is only those contours below the phreatic surface that are considered). The figures also show the flux quantities across lines at both the near and far levees for each seepage condition.

The analysis results are also summarized for each case on Table 2.3.2. The table presents the following:

- The average total head (in feet) in the sand aquifer at the near levee centerline.
- The seepage flux (in gpm per foot of levee) at the near levee centerline. Where two flux quantities are given for the pumping condition, each flux rate represents the flow from one side of the line of pumps. The total pumping rate is the sum of the two values.
- The average total head (in feet) in the sand aquifer at the far levee centerline.
- The flux quantity (in gpm per foot of levee) at the near far centerline.
- The water table level at the far toe of the far levee (near the ditch).
- The corresponding pump rate for individual interceptor wells spaced at 160 feet (for pumping conditions only).

For the flux quantities, flows away from the slough within the sand aquifer (like those found in existing conditions) are considered positive and those flows toward the slough are considered negative. This sign convention was adopted to easily identify reversals in flow directions on Table 2.3.2.

Webb Tract Station 630+00. This cross-section was considered to be a critical seepage condition for Webb Tract, as the adjacent island levee is only about 400 feet away (levee center to levee center across Fisherman's Cut). The total head within the sand aquifer at each levee under existing seepage conditions is about –15 ½ feet, as shown on Figure 2.3.2. The existing conditions diagram shows a significant drop in total head within the channel silt, indicating the importance of the channel silt's influence on the seepage rates under the levees (see also the discussion in Section 2.3.2 under "Analysis Sections" regarding evidence of the existence of channel silt). The calculated water table at the far toe of the far levee is at about elevation –17 feet, which is just below the drainage ditch.

Under full reservoir conditions with no pumping at the interceptor wells, there is a seven-foot increase in the total head beneath the far levee and upward flow into the neighboring island, as shown on Figure 2.3.3. In addition, a review of the exit gradients near the drainage ditch on the land side of the far levee indicates that gradients over 0.6 exist at the ground surface. Under these gradients, there would likely be sand boils and piping of levee material on the neighboring island.

Under full reservoir conditions with pumping at the interceptor wells, the minimum head at the pump needed to retain pre-reservoir conditions at the adjacent island is about –15 feet. This corresponds to an average pumping rate along the well line of 0.076 gpm per foot of levee, or about 12 gpm for wells spaced at 160 feet.

Webb Tract Station 260+00. This second cross section for Webb Tract was considered to be a less critical seepage condition than that at Station 630+00, as the adjacent island levee on Mandeville Island is about 1200 feet away (center to center). The total head within the sand aquifer at each levee under existing seepage conditions is about –9 to –10 feet, as shown on Figure 2.3.4. The water table at the far toe of the far levee is at about elevation –9 feet, which is about the level of the drainage ditch.

Under full reservoir conditions with no pumping at the interceptor wells, there is only a ½ -foot increase in the total head beneath the far levee, which is hardly enough to cause a change in flow into the neighboring island, as shown on Figure 2.3.5. Nevertheless, in order to maintain a no-change condition, the minimum head at the pump needed to retain pre-reservoir conditions at the adjacent island is about –10 feet. This corresponds to an average pump rate along the well line of 0.066 gpm per foot of levee, or about 10-½ gpm for wells spaced at 160 feet. The required pump rate is slightly smaller that that found at Webb Tract Station 630+00, which is a more critical case with a narrower slough. The smaller pump rate to maintain the required head in the adjacent island is due to the greater length of the sand aquifer beneath the slough at Station 260+00 through which the groundwater must flow to reach the adjacent island.

Bacon Island Station 220+00. This cross section was considered to be a critical seepage condition for the Bacon Island, as the adjacent island levee on Mandeville Island is only about 450 feet away (center to center). The total head within the sand aquifer at each levee under existing seepage conditions is about –14 feet, as shown on Figure 2.3.6. The existing conditions diagram shows a significant head drop within the channel silt (as it did at Webb Tract), indicating the importance of the channel silt's influence on the seepage rates under the levees. The water table at the far toe of the far levee is at about elevation –17 feet, which is about the bottom of the drainage ditch.

Under full reservoir conditions with no pumping at the interceptor wells, there is a four-foot increase in the total head beneath the far levee, as shown on Figure 2.3.7. However, the phreatic surface still lies beneath the ground surface on the adjacent island (no surface flow). Under full reservoir conditions with pumping at the interceptor wells, the minimum head at the pump needed to retain pre-reservoir conditions at the adjacent island is about –14 feet. This corresponds to an average pump rate along the well line of 0.053 gpm per foot of levee, or about 8-½ gpm for wells spaced at 160 feet.

Bacon Island Station 665+00. This second cross section for Bacon Island at Station 665+00 was considered to be a less critical seepage condition than that at Station 220+00 because of the presence of a 16-foot thick layer of clay above the sand aquifer and the greater distance between levees. Under existing seepage conditions the total head within the sand aquifer at each levee was about -14 feet, as shown on Figure 2.3.8. The water table at the far toe of the far levee on Woodward Island was at about elevation –9 feet, which is about the level of the drainage ditch.

Under full reservoir conditions with no pumping at the interceptor wells, there is a five-foot increase in the total head beneath the far levee, as shown on Figure 2.3.9. The phreatic surface rises to just below the ground surface on the adjacent island. Under full reservoir conditions with pumping at the interceptor wells, the minimum head at the pump needed to retain pre-reservoir conditions at the adjacent island is about –14 feet. This corresponds to an average pump rate along the well line of 0.033 gpm per foot of levee, or about 5 gpm for wells spaced at 160 feet.

Sensitivity Analyses. Three sensitivity analyses were conducted to evaluate the change in seepage conditions when changes occur in the hydraulic conductivities of the channel silt and aquifer sand, and in the thickness of the peat on each island. Webb Tract Station 630+00 was used for all sensitivity analyses, and the specific variation were as follows:

- Increasing the channel silt hydraulic conductivity from $1x10^{-6}$ cm/s to $5x10^{-6}$ cm/s.
- Increasing the aquifer sand hydraulic conductivity from $1x10^{-3}$ cm/s to $5x10^{-3}$ cm/s.
- Decreasing the peat thickness over the islands from six feet to three feet.

When increasing the channel silt hydraulic conductivity from $1x10^{-6}$ cm/s to $5x10^{-6}$ cm/s, a smaller head loss occurs within the silt layer and water levels increase throughout the aquifer, as shown on Figure 2.3.10. When compared to the baseline case (Figure 2.3.2), the head in the aquifer sand at the levees increased from –15 ½ feet to –10 ½ feet. Flows beneath the levees also increase from 0.0066 gpm per foot of levee for the baseline case, to 0.0159 gpm per foot of levee for the case using a higher silt hydraulic conductivity. So for an increase in the channel silt's hydraulic conductivity by a factor of five, the flow rates increased by a factor of 2 ½.

This model using a higher hydraulic conductivity for the channel silt also shows that the phreatic surface is above the ground surface (indicating flooding) on both islands under existing conditions, which is not seen in the field. For this reason it is felt that this modeled condition is not representative of actual conditions. As shown on Figure 2.3.11, this model shows a similar increase in the total head distribution for the condition of a full reservoir both with and without pumping, when compared to the baseline cases. The pump rate required to retain pre-reservoir conditions for this case is comparable to that found for the baseline case (11gpm vs. 12 gpm for wells at 160 feet). Therefore, when considering the performance of the well interceptor system, the project is not sensitive to a change by factor of five in the channel silt hydraulic conductivity.

When increasing the aquifer sand hydraulic conductivity from $1x10^{-3}$ cm/s to $5x10^{-3}$ cm/s (which is approximately the value determined from the McDonald Island drawdown test), the total head under each levee decreases from –15 ½ feet (baseline case) to –18 ½ feet, as shown on Figure 2.3.12. For the condition of a full reservoir with no pumping (Figure 2.3.13), the total head within the aquifer at each levee is about one foot lower than that found in the base case, and the flow rate beneath each levee increases by a factor of about four. The pump rate at the interceptor wells necessary to achieve conditions at the far levee similar to those found during pre-reservoir conditions is about three times the pump rate for the base case (38 gpm vs. 12 gpm for wells at

160 foot spacing). This analysis illustrates the dependency of the required pump rates on the hydraulic conductivity.

When decreasing the estimated peat thickness over the reservoir and neighboring islands from six feet to three feet, there was little affect on the total head contours within the sand aquifer, as shown on Figure 2.3.14. However the model also shows the phreatic surface is above the ground surface on both islands (indicating flooding) under existing conditions, which is not seen in the field. For this reason it is felt that this model is not representative of actual conditions. The thinning of the peat also has only a minimal affect on the total head values and pump rates for the condition of a full reservoir with and without pumping at the interceptor wells, as shown on Figure 2.3.15. The thinning of the peat layer resulted in an increase in the required pump rate from 12 to 13 gpm for wells at 160-foot spacing. Overall, the influence of the peat layer thickness on seepage conditions within the section is considered minimal.

Borrow Areas. In order to determine the effect of the proposed borrow areas on the seepage conditions within the sand aquifer, a model was constructed in which the 500-foot wide and 40-foot deep borrow area was added to the model of Webb Tract at Station 630+00. The borrow area was located about 400 feet from the toe of the levee as shown on Figure 2.3.16. The seepage condition of full reservoir with pumping at the interceptor wells was considered for the comparison, the results of which are detailed on Table 2.2. The construction of the borrow area 400 feet from the levee has little to no effect on the total head conditions within the aquifer near the levees, or on the required pump rate at the interceptor well when compared to baseline estimates. To follow US Army Corps of Engineers requirements (USACE, 1978), the borrow areas should be constructed at least 800 feet from the levee toe. This seepage analysis shows that a borrow area constructed 800 feet from the levee will not have a detrimental impact on the seepage conditions or on operation of the well interceptor system. Therefore, there is no need to "seal" the borrow excavation by placing the excavated silt overburden back into the excavation.

2.3.5 Summary of Findings

The seepage analyses conducted for four cross sections taken along the Webb Tract and Bacon Island levees show considerable variations in the existing flow conditions and those anticipated following filling of the proposed reservoirs and installation of the interceptor well system. These variations in subsurface stratigraphy and levee configuration between adjacent islands result in varying total head conditions and flow rates within the sand aquifer as well as the required pump rate. However, for all of the cases considered, a properly functioning interceptor well system can be used to minimize the effects of the proposed reservoirs on adjacent islands, including the potential for rises in the ground water table or flooding.

Seepage analyses show that the proposed reservoir at Webb Tract may increase the water table beneath the levee at adjacent islands from ½ to 7 feet at the sections analyzed, and that flooding may occur in the neighboring islands in the absence of pumping at the interceptor well system. In order for the well system to intercept the reservoir-induced seepage and maintain existing seepage conditions beneath the levees at adjacent islands, pump rates of 10 to 12 gpm (for wells at 160-foot spacing) would be required. However, previous studies have shown variations in the hydraulic conductivity of the sand aquifer to levels five to six times those used in these analyses. As shown in the sensitivity analyses, such a variation in the sand's hydraulic conductivity would result in an increase in the required pump rate to 50 to 60 gpm for wells spaced at 160 feet.

Seepage analyses show that the proposed reservoir at Bacon Island may increase the water table beneath the levee at adjacent islands from about 5 feet at the sections analyzed. In order for the well system to intercept the reservoir-induced seepage and maintain existing seepage conditions beneath the levees at adjacent islands, pump rates of 5 to 8-½ gpm (for wells at 160-foot spacing) would be required. As mentioned above, possible variations in the sand aquifer's hydraulic conductivity may result in an increase in the required pump rate to up to five times these values.

The proposed borrow area locations of 400 feet or farther from the existing levees on the reservoir islands should have little or no influence on the seepage conditions beneath the island levees. To follow US Army Corps of Engineers requirements, the borrow areas should be constructed at least 800 feet from the levee toe. The seepage analysis shows that a borrow area constructed 800 feet from the levee will not have a detrimental impact on the seepage conditions or on operation of the well interceptor system.

For both Webb Tract and Bacon Island, the interceptor well system should extend to the bottom of the sand aquifer. The pumping well should be screened over the entire length of the aquifer to achieve the required drawdown at the well, and the pumps should efficiently handle the required pump rate. The proposed spacing of 160 feet between pumping wells seems to be adequate; however, more optimum spacings and pump rates may be found for each levee section during the final design of the project. Following detailed investigations of subsurface conditions, adjustments in the well interceptor system design will be required to accommodate varying conditions, ranging from areas where little or no pumping may be needed (e.g., next to the San Joaquin River) to areas where pumping rates may be much higher than is typical (e.g., along localized gravelly portions of the aquifer).

The interceptor well concept generally appears to be able to mitigate seepage problems induced by the proposed reservoirs; however, proper design and construction will be key to the success of the interceptor well system. The water table level on the adjacent islands is considered to be an important indicator of impacts detrimental to those islands, as a significant rise in the ground water table may affect agricultural operations and production rates. The wells will have to be maintained at regular intervals to ensure their effectiveness. Further, the proposed observation wells that will be installed on the adjacent island levees must be monitored consistently to help ensure that the interceptor wells are operated at the pump rate that minimizes potential impacts on neighboring islands. (Estimated effects of pump outages are discussed in Section 2.5.2.)

2.4 EVALUATION OF MONITORING SYSTEM AND PROCEDURES

DW proposed a seepage monitoring system for the detection of seepage in the neighboring islands due to the implementation of the project (HLA 1991c, 1991d, 1992c; Hultgren 1997a, 1997b). Significance standards were proposed by DW to evaluate the seepage monitoring data for the determination of implementing seepage control measures.

This section presents a review of the proposed seepage monitoring system, the significance standards and the seepage control measures (Section 2.4.1); and an evaluation of the adequacy of the proposed seepage monitoring system and the significance standards (Section 2.4.2).

2.4.1 Proposed Seepage Monitoring System and Significance Standards

Seepage Monitoring System

At least one year prior to first filling of the DW reservoir islands (Webb Tract and Bacon Island), approximately 104 groundwater monitoring wells are recommended by DW for installation on neighboring islands. About 77 of the wells are seepage monitoring wells that will be sited directly opposite the DW reservoir islands. The other about 27 wells are background monitoring wells to provide groundwater variations at locations that are not expected to be impacted by the project related seepage. Conceptual locations of the proposed monitoring wells are shown in Figure 2.4.1. The purpose of the monitoring wells is to provide an early detection of seepage caused by the project.

Since the majority of the seepage into the neighboring islands is likely to occur through the most permeable sand layer (referred to as "deep seepage" in the DW reports), the piezometers will be screened in the sand layer. The following guidelines were used for the seepage monitoring piezometer spacing:

- A spacing of 1,500 to 2,000 feet on neighboring islands to closely monitor a continuous sand aquifer that underlies both the DW project and neighboring islands,
- A minimum spacing of 1,000 feet at critical sections, and
- A maximum spacing of 4,000 feet at other sections.

The background piezometers will be located in neighboring island locations which will not be impacted by the project related seepage.

The piezometers will be instrumented with pressure transducers, which will be connected with programmable data loggers. The data loggers will be programmed to collect water levels hourly, and the hourly water level readings will be averaged to compute a daily mean for each piezometer. Water levels will be concurrently recorded in the rivers and sloughs near the project islands.

Significance Standards

DW proposed seepage performance standards or significance standards to identify net seepage increases in the neighboring islands attributable to the reservoir islands. The data collected from the monitoring network will be used for application of the significance standards. If the data show exceedance of the significance standards, DW proposes to trigger seepage control measures to control the increased seepage.

Data collection from the piezometers will commence at least one year prior to filling of reservoirs. The data collected during this period will form the "historic" conditions at these locations. During filling and storage, water levels in monitoring wells on neighboring islands will be compared to the historical data and to the background data collected from the background wells. The purpose of the comparison with historic data is to evaluate whether a correlation exists between the piezometric levels and the reservoir filling and storage. The comparison with the background data is to check whether the variations observed are occurring throughout the Delta or only near the reservoir islands.

The proposed significance standards are presented below:

Significance Standards Proposed by Delta Wetlands

Standard	Condition 1		Condition 2		Condition 3
One Monitoring Well	Groundwater level in monitoring well > historic mean groundwater level + two standard deviation +1 foot	*and*	Increased groundwater level in monitoring well correlates with reservoir filling	*and*	Level corrected for current variations in background groundwater level
3 or More Contiguous Monitoring Wells	Groundwater level in monitoring wells > historic mean groundwater level + two standard deviation + 0.25 foot	*and*	Increased groundwater level in monitoring wells correlates with reservoir filling	*and*	Level corrected for current variations in background groundwater level

Note: All three conditions must be met simultaneously to trigger seepage control measures.

Hypothetical patterns related to seepage performance standards for a group of three wells are shown in Figure 2.4.2. This figure shows three scenarios: no reservoir related seepage case (Case I), seepage increase not attributable to the project (Case II), and seepage increase caused by the project (Case III). In Case II, mean water levels in three wells exceed the significance standards, but mean background water levels in background piezometers show a corresponding increase, indicating a regional water level rise not caused by the project. In Case III, seepage increase is attributable to the project because the background piezometers do not show a corresponding increase.

Seepage Control Measures

If seepage increase is detected as identified in Case III, DW will undertake measures to control the seepage. The primary means to control seepage is pumping from seepage interceptor wells placed on the reservoir islands levee. If the interceptors wells alone, even with increased pumping, may not be enough to control seepage, DW proposes to install additional interceptor wells, install relief wells (wells that passively relieve elevated water pressures in an aquifer), and take other methods acceptable to the landowners and reclamation districts. If DW is unable to control project related seepage and it cannot work out a satisfactory solution with the landowners and the reclamation district, DW proposes to lower the reservoir levels to avoid the impacts. In the most extreme case, DW proposes to completely eliminate the reservoir operations (Hultgren 1997a). The report indicates that the significance standards have been approved by the Seepage Review Committee. However, hearing testimony and oral statements at meetings contradict this.

2.4.2 Comments on Adequacy of Seepage Monitoring System and Significance Standards

Seepage Monitoring System

DW proposes to monitor the achievement of the no-net-seepage condition to neighboring islands by two sets of monitoring wells, seepage monitoring wells and background wells. Seepage monitoring wells are proposed to be placed on the crest of the levees of islands located across sloughs or channels from the DW reservoir islands. Background well are proposed to be located typically on the opposite sides of the neighboring islands. The proposed system of monitoring wells and background wells is shown in Figure 2.4.1.

To review the effectiveness of the proposed background wells, we evaluated the relationship between water levels measured in monitoring wells spaced some distance apart from each other. Existing monitoring wells located in various islands neighboring the reservoir islands were received and compared for similarity in trend and groundwater elevation in time. The objective of the comparison is to determine if all the wells located in an individual island show similar groundwater level increase and decrease trends before project implementation. Groundwater monitoring data collected as part of the ongoing background groundwater monitoring (see Section 2.2 for details) were used in the comparison. The groundwater monitoring data are presented in HLA 1995c and 1995d; data for one monitoring well (BA-6) are reproduced in Figure 2.4.3 as a sample of the data reviewed.

The observations from the review of 22 monitoring wells within the project islands or the neighboring islands indicate:

- The recorded water elevations in wells within the same island are different. The differences in water elevation within each island vary from 2 feet (Bethel Island, wells BE-11 and BE-12) to as much as 12 feet (Venice Island, wells VN-32 and VN-34).
- Most of the wells show a cyclical trend in groundwater elevation, which is higher in the winter seasons. However, there are some exceptions where no particular trends were noted, such as at Bouldin Island (well BO-28), Holland Tract (well HO-2), Palm Tract (well PA-30), Venice Island (wells VN-32 and VN-33). At Woodward Tract there exists a trend but it is out of phase from the other wells (peaks in water table do not occur at the same time).
- Because of the lack of correlation in groundwater elevations and seasonal trends, it is recommended that a revised background well system be considered in each neighboring island. This will allow accounting for the local variation of groundwater level within each adjacent island. Multiple background monitoring wells will also offer the opportunity to account for groundwater changes due to local pumping operations for various farming needs within each neighboring island.
- This system of background wells can be composed of the proposed background wells by Delta Wetland, supplemented by shallow background wells (10 to 20 feet deep) installed across each neighboring island to monitor the trend of groundwater away from the reservoir islands. These additional background wells can be placed a half-mile to one mile apart. The specific location and spacing should be finalized in the design phase of the project based on groundwater conditions in each neighboring island.

Significance Standards

The significance standards established by DW to trigger initiation of seepage control measures (i.e., pumping of the seepage monitoring wells in the first place) are summarized in Section 2.2. They use three simultaneous conditions to identify triggering conditions: exceedance of water level in one or several monitoring wells of significance levels, correction for background water levels, and correlation with reservoir filling. All three conditions must be satisfied to actuate the trigger. The three conditions appear appropriate to identify project-related seepage. Provided that background wells are installed as noted previously, the significance standards are the only condition of concern.

Use of one year to establish a reference base of water levels in the seepage monitoring well and background wells does not appear to be long enough. We recommend a base of three years, to optimize the probability that realistic variations in the water levels with the seasons and with relatively dry and wet years are established. Considering that construction of the improvements to the reservoir island levees will likely take more than three years, this condition should be easy to satisfy. The three-year base should be used for the background wells and at least a portion, say half, of the seepage monitoring wells.

Use of the mean plus two standard deviations (to include about 95 percent of the data points) appears reasonable in the calculation of the significance standard. (There would be too many "false alarms" if a smaller value were used.) We also recommend the use of the straight line running mean rather than the simple harmonic (sine function).

Use of one foot of "leeway" in a single monitoring well is judged to be too high. This judgment is based primarily on the results of the seepage analyses, which show that the difference in water heads in the aquifer below the toe of the near levee of the adjacent island is only four feet, when comparing full reservoir conditions with and without pumping. Further, as discussed earlier, there is a time lag involved between the onset of pumping and the time there is an effect on the water head at the toe of the adjacent island's levee. This lag time is expected to be on the order of one day, as discussed in Section 2.5.2. It is our judgment that undesirable seepage effects in the adjacent island could start with a one-foot rise in the water table. Considering that the one-foot margin includes the two standard deviations in the monitoring well reading, the "leeway" and the time lag effect, it is our judgment that the "leeway" should be limited to 0.5 foot for a single well. The leeway of 0.25 foot for the average of three wells appears acceptable.

2.4.3 Conclusions and Recommendations

- The proposed system of seepage monitoring wells appears appropriate.
- Background wells shall include both those conceptually proposed by DW and additional rows of shallow monitoring wells across adjacent islands.
- Use more than one well for background data collection for each row of seepage monitoring wells.
- Use at least three years of data to establish reference water levels in the background wells and at least one half of the seepage monitoring wells.
- Use running straight-line mean from monitoring well data in the application of the significance criteria.

- Reduce the "leeway" for a single monitoring well to 0.5 foot; 0.25-foot leeway for the average of three wells is acceptable.
- Other data (e.g., undesirable seepage effects such as reported impacts on agriculture in adjacent islands, or results of well effectiveness tests as discussed in Section 2.5) may be used in conjunction with significance standards to justify deviations from the standards.
- The significance standards should be reviewed periodically after startup of reservoir operations to validate their utility; suggested times of reevaluation are after 2, 5 and 10 years of operation.

2.5 LONG-TERM MONITORING OF SYSTEM PERFORMANCE

2.5.1 Long-Term Reliability of Proposed Well System

The main components of the proposed interceptor well system will be the wells, the collector piping, and the power supply and controls. Long-term reliability of the system will depend on the functioning of all these constituents.

It is important that the individual wells making up the interceptor well system are carefully designed and constructed as long-term production wells. Specifically, this will involve design of the well screen and surrounding gravel pack to be done to accommodate the grain sizes of the aquifer, in accordance with applicable criteria. Subsequently, the wells must be constructed and developed appropriately. Further, the perforated section of the well casing should stay permanently submerged (i.e., should not extend above the elevation of the deepest expected drawdown of the water table), to minimize the possibility of fouling of the screen by organic growths. Over time, regular well and pump maintenance must be performed to ensure continued optimal functioning of the wells. It will be useful in this connection if the individual wells were equipped with flow meters, such that any dropoff in output could be identified.

The collector piping is unlikely to be the source of any system reliability problems.

The electrical power supply may be interrupted at times. It is expected that a power outage not exceeding a few hours will have no significant effect. It may be worthwhile, in final design, to evaluate the likely power outages and their consequences on seepage control, and consider if provision of standby generators is advisable.

The control system will include the piezometers, their monitoring, transmission and evaluation of data, and the tie-in between the monitoring and pumping, i.e., the application of the "significance criteria." It is expected that the piezometer reading and transmission and evaluation of data will be implemented in such a way, and with sufficient manual checks, that these items will not significantly impact reliability.

In summary, therefore, long-term operability of the individual wells and reliability of power supply are expected to be the main potential sources of inadequate system performance. We believe that rigorous well O&M and consideration of standby power will provide high likelihood of long-term system reliability.

The possibility that the extraction wells could cause long-term loss of fines in the vicinity of the well, which can have potential stability and settlement implications, is discussed in Section 3.11.

2.5.2 Estimated Effects of Pump Outages

The seepage analyses presented in this report were made for steady-state conditions; i.e., for a condition expected to last sufficiently long that transient effects are not present. Rough hand calculations suggest that the "travel time" from the pumps to the land side toe of the adjacent island is at least one day. Therefore, a pump outage would be felt in the adjacent island at least a day later. Correspondingly a restart of pumping is expected to have a similar time lag in its effect starting to be felt.

This time estimate confirms the judgment that a pump outage, for instance due to a power failure, of a few hours would have no significant effect. An outage extending to day or more would be expected to cause a rise in the groundwater table in the adjacent islands. Another possibility is that one or several adjacent pumps may not be performing as expected. This would not be known (absent individual flow meters on the wells and their periodic reading) until a piezometer showed an unusual rise in the water table at a location. With piezometers spaced a minimum of 1000 feet apart, such a lack of performance might not be noticed for quite some time if the affected wells are located between piezometers. It is possible that the effects of such an event would be identified on the ground before they were detected by the piezometers. Installation of individual flow meters on wells and their periodic monitoring would minimize the potential of this occurrence.

2.5.3 Monitoring Procedures to Detect and Respond to Outages

The needed monitoring procedures follow from the above discussion. The principal monitoring method to detect the effects of poorly performing well(s) is the periodic reading of the piezometers, ideally by remote-operating and transmitting means. To guard against the effects of partial or complete outages of individual or groups of wells, the output of individual wells (by permanently or temporarily installed flow meters) should be monitored periodically. In areas considered critical, closer spacing of piezometers to minimize the possibility of occurrence of high water tables between piezometers may also be considered.

In the event of partial or system-wide power outages (with stored water in the reservoir), the responsible reservoir operators should be notified immediately by automatic alert. If the outage should last more than a few hours, appropriate notice should be given to adjacent island operators. Should the outage last more than a few hours, adjacent islands should be patrolled for potentially undesirable seepage effects, and appropriate remedial measures (including reservoir lowering in the extreme) should be taken if such effects are apparent.

2.6 WATER DIVERSIONS FROM ADJACENT CHANNELS THROUGH THE INTERCEPTOR WELLS

During certain conditions in the reservoir islands and adjacent channels, the pumping from the interceptor well system may divert water from the channels onto the reservoir islands. Based on the results of the seepage analyses performed for Webb Tract and Bacon Island, which are described in detail in Section 2.3, this report contains an assessment of the amount of water that could be inadvertently diverted onto the reservoir islands through operation of the interceptor well system and direct seepage.

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The pump rates estimated for the interceptor well system that would be required to avoid the effect of reservoir induced seepage and described in Section 2.3. This pump rate would create seepage conditions beneath the adjacent levee that are approximately equal to those seen under pre-reservoir (without-project) conditions. The estimated required pump rates to achieve pre-reservoir conditions at the adjacent island levee at each of the four sections considered are summarized below:

Case	Flux Away from Slough, Toward Reservoir Island (gpm per foot of levee)		
	Existing Conditions	Full Reservoir with No Pumping	Full Reservoir with Required Pumping
1. Webb Tract, Station 630+00	0.0066	-0.0167	0.0061
2. Webb Tract, Station 265+00	0.0163	-0.0076	0.0174
3. Bacon Island, Station 220+00	0.0080	-0.0069	0.0089
4. Bacon Island, Station 665+00	0.0010	-0.0057	0.0061

For Cases 1, 2 and 3, when the required pumping rate is used, the flux from the slough toward the reservoir island is about the same as the flux seen during existing conditions. For these cases, the pumps are drawing no more water from the slough than is flowing under existing conditions. The exception is in Case 4, in which some additional flux from the slough flows towards the well system as it draws down the water level to achieve the required conditions at the neighboring island. The reason for this additional pumping effort appears to be the presence of the upper clay layer near Station 665+00 at Bacon Island.

To further illustrate an example of how the interceptor wells would capture water from the adjacent channel, the seepage model developed for Webb Tract at Station 630+00 was used. As shown in Figure 2.6.1, a required pump rate of 0.0759 gpm per foot of levee (or 12 gpm per well spaced at 160 feet) was found to correspond to a drawdown of –15 feet at the pumping well. At this rate the average total head (-17 feet) and flow rate (0.007 gpm per foot) beneath the adjacent island's levee are approximately equal to the values seen under existing conditions (without the project). Should the pump rate at the interceptor well be increased 25% to 0.0955 gpm per foot (or 15 gpm for wells spaced at 160 feet), the drawdown at the well increases to –20 feet (see Figure 2.6.1). As shown on the figure, the flux rate from the slough side of the pump would increase from 0.0061 to 0.0128 gpm per foot. In addition, the average total head in the sand aquifer beneath the adjacent levee would drop from -15 ½ to -17 ½ feet, and the flux rate there (from the slough toward the island) would decrease from 0.0071 to 0.0033 gpm per foot.

A method for monitoring water seeping onto the reservoir islands from the adjacent slough could include both monitoring of pump rate at the interceptor well system as well as monitoring piezometer levels on the adjacent island's levee to watch for significant changes from baseline values. To account for the influence of seasonal changes in the existing seepage conditions beneath the adjacent island levees, the monitoring program on the adjacent levees should be established well in advance of reservoir filling to develop the record of the baseline conditions against which new readings can be compared. Discussions of the adequacy of the monitoring system and the proposed significance criteria are presented in the preceding sections.

2.7 POTENTIAL SETTLEMENTS CAUSED BY FILLING AND EMPTYING RESERVOIR ISLANDS

There will be some additional island surface settlement associated with initial and subsequent filling and emptying of the reservoir islands. Conceptual consideration of the mechanisms that would lead to additional settlements leads us to the conclusion that additional settlements are expected to be nominal, of the order of one additional foot of settlement. This amount is less than would be expected from continued use of the islands for agriculture, which would over time lead to essentially complete oxidation of the peat. This process would correspond to as much as 15 feet of additional settlement on Webb Tract and 10 feet of additional settlement on Bacon Island. In fact, flooding of islands has been proposed as one method to minimize further oxidation of peat and associated subsidence in the Delta islands.

C-063408

Table 2.2.1
Permeability of Soils Used in Prior Seepage Analysis

Material	Vertical Permeability Ky (cm/s)	Horizontal Permeability Kx (cm/s)	Ky/Kx	Ref.
Existing Sandy Fill (with clay and peat)	1×10^{-5}	1×10^{-4}	0.1	1
Existing Clayey Fill (Bay Mud)	1×10^{-7}	1×10^{-6}	0.1	1
Peat	1×10^{-6}	1×10^{-4}	0.01	1, 2
Silty Sand	1×10^{-4} to 5×10^{-4}	1×10^{-3} 5.4×10^{-3} to 6.4×10^{-3}	0.1 to 0.5	1 3
Sand with gravel		1.4×10^{-1}	1	3
Clay/Silt (at bottom of channel)	1×10^{-6}	1×10^{-6}	1	1
Planned Fill (sand)	1×10^{-4}	1×10^{-3} 1.1×10^{-3}	0.1	1 2

1 - HLA 1989. Preliminary Geotechnical Investigation for the Delta Wetlands Project. Feb 15, 89. pp -13.
2 - HLA 1992. Geotechnical Investigation & Design of the Wilkerson Dam on Bouldin Island. May 27, 92. pp -16.
3 - HLA 1991. Interceptor Well Modelling for the Delta Wetlands project. pp A-7 to A-8.

Notes: In ref. 1, vertical permeabilities were measured, and horizontal permeabilities were estimated.
It is not clear if the permeabilities from ref. 2 were measured or not (this ref gives only horizontal permeability).
Permeabilities from ref. 3 were measured using McDonald Island pump tests data (this ref. gives only horizontal permeability).

Table 2.3.1
Soil Properties Used in Seepage Analysis of Four Cross Sectional Models

Cross Section	Soil Layer	Approximate Soil Layer Thickness (feet)	Horizontal Hydraulic Conductivity K_x (cm/s)	Vertical Hydraulic Conductivity K_y (cm/s)
Webb Tract Sta. 260+00	Fill Material (Clay with Peat and Sand)	12	1×10^{-4}	1×10^{-5}
	Peat	25	1×10^{-4}	1×10^{-5}
	Sand	46	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	Channel Silt	3	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}
Webb Tract Sta. 630+00	Fill Material (Sand)	10	1×10^{-4}	1×10^{-5}
	Fill Material (Clay)	5	1×10^{-6}	1×10^{-6}
	Peat	15	1×10^{-4}	1×10^{-5}
	Sand	50	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	Channel Silt	3	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}

Table 2.3.1

Soil Properties Used in Seepage Analysis
of Four Cross Sectional Models (continued)

Cross Section	Soil Layer	Approximate Soil Layer Thickness (feet)	Horizontal Hydraulic Conductivity K_x (cm/s)	Vertical Hydraulic Conductivity K_y (cm/s)
Bacon Island Sta. 220+00	Fill (Sand and Clay)	7	1×10^{-4}	1×10^{-5}
	Peat	30	1×10^{-4}	1×10^{-5}
	Sand	20	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	Channel Silt	3	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}
Bacon Island Sta. 665+00	Fill Material (Peat)	20	1×10^{-4}	1×10^{-5}
	Upper Clay	16	1×10^{-6}	1×10^{-6}
	Sand	19	1×10^{-3}	1×10^{-4}
	Lower Clay	--	1×10^{-6}	1×10^{-6}
	Channel Silt	3	1×10^{-6}	1×10^{-6}
	New Fill (Sand)	Varies	1×10^{-3}	1×10^{-3}

Table 2.3.2
Seepage Analysis Results

Case	Description	Figure Number	Head in Sand at Near Levee CL (feet)	Flow at Near Levee CL (gpm/ft)	Head in Sand at Far Levee CL (feet)	Flow at Far Levee CL (gpm/ft)	Water Table At Far Toe of Far Levee (feet)	Pumping Rate Required for Wells at 160' (gpm)
1	Webb Tract - Station 630+00 Existing Conditions	2.3.2	-15 ½	0.0066	-15 ½	0.0067	-17	NA
2	Webb Tract - Station 630+00 Full Reservoir w/ no Pumping	2.3.3	- ½	-0.0167	-8 ½	0.0208	-13	NA
3	Webb Tract - Station 630+00 Full Reservoir w/ Pumping	2.3.3	-15	0.0[illegible] (pumping)	-15 ½	0.0071	-17	12
4	Webb Tract - Station 260+00 Existing Conditions	2.3.4	-10	0.0163	-9	0.0142	-9	NA
5	Webb Tract - Station 260+00 Full Reservoir w/ no Pumping	2.3.5	-3 ½	-0.0076	-8 ½	0.0167	-9	NA
6	Webb Tract - Station 260+00 Full Reservoir w/ Pumping	2.3.5	-10	0.0660 (pumping)	-9	0.0149	-9	10 ½

Table 2.3.2 continued

Case	Description	Figure Number	Head in Sand at Near Levee CL (feet)	Flow at Near Levee CL (gpm/ft)	Head in Sand at Far Levee CL (feet)	Flow at Far Levee CL (gpm/ft)	Water Table At Far Toe of Far Levee (feet)	Pumping Rate Required for Wells at 160' (gpm)
7	Bacon Island - Station 220+00 Existing Conditions	2.3.6	-14	0.0080	-14	0.0078	-17	NA
8	Bacon Island - Station 220+00 Full Reservoir w/ no Pumping	2.3.7	2	-0.0069	-9 ½	0.0118	-13	NA
9	Bacon Island - Station 220+00 Full Reservoir w/ Pumping	2.3.7	-14	0.0527 (pumping)	-14	0.0076	-17	8 ½
10	Bacon Island - Station 665+00 Existing Conditions	2.3.8	-14	0.0010	-14	0.0010	-9	NA
11	Bacon Island - Station 665+00 Full Reservoir w/ no Pumping	2.3.9	- ½	-0.0057	-9	0.0049	-9	NA
12	Bacon Island - Station 665+00 Full Reservoir w/ Pumping	2.3.9	-14	0.0333 (pumping)	-14	0.0011	-9	5

Table 2.3.2 continued

Case	Description	Figure Number	Head in Sand at Near Levee CL (feet)	Flow at Near Levee CL (gpm/ft)	Head in Sand at Far Levee CL (feet)	Flow at Far Levee CL (gpm/ft)	Water Table At Far Toe of Far Levee (feet)	Pumping Rate Required for Wells at 160' (gpm)
SENSITIVITY ANALYSES								
S1	Webb Tract - Station 630+00 Existing Conditions (Channel Silt at $5x10^{-6}$ cm/s)	2.3.10	-10 ½	0.0159	-11	0.0165	-13	NA
S2	Webb Tract - Station 630+00 Full Reservoir w/ no Pumping (Channel Silt at $5x10^{-6}$ cm/s)	2.3.11	+2	-0.0134	-7.5	0.0242	-13	NA
S3	Webb Tract - Station 630+00 Full Reservoir w/ Pumping (Channel Silt at $5x10^{-6}$ cm/s)	2.3.11	-10	0.0681 (pumping)	-10 ½	0.0168	-14	11
S4	Webb Tract - Station 630+00 Existing Conditions (Aquifer Sand at $5x10^{-3}$cm/s)	2.3.12	-18 ½	0.0085	-18 ½	0.0086	-18 ½	NA
S5	Webb Tract - Station 630+00 Full Reservoir w/ no Pumping (Aquifer Sand at $5x10^{-3}$cm/s)	2.3.13	-1 ½	-0.0702	-9 ½	0.0758	-11	NA

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Table 2.3.2 continued

Case	Description	Figure Number	Head in Sand at Near Levee CL (feet)	Flow at Near Levee CL (gpm/ft)	Head in Sand at Far Levee CL (feet)	Flow at Far Levee CL (gpm/ft)	Water Table At Far Toe of Far Levee (feet)	Pumping Rate Required for Wells at 160' (gpm)
S6	Webb Tract - Station 630+00 Full Reservoir w/ Pumping (Aquifer Sand at $5x10^{-3}$cm/s)	2.3.13	-18 ½	0.2384 (pumping)	-18 ½	0.0092	-18 ½	38
S7	Webb Tract - Station 630+00 Existing Conditions (Peat Thickness reduced from 6 ft to 3 ft)	2.3.14	-15	0.0063	-15	0.0069	-16	NA
S8	Webb Tract - Station 630+00 Full Reservoir w/ no Pumping (Peat Thickness reduced from 6 ft to 3 ft)	2.3.15	1	-0.0169	-8 ½	0.0208	Above -13	NA
S9	Webb Tract - Station 630+00 Full Reservoir w/ Pumping (Peat Thickness reduced from 6 ft to 3 ft)	2.3.15	-15	0.0819 (pumping)	-15	0.0070	-16	13

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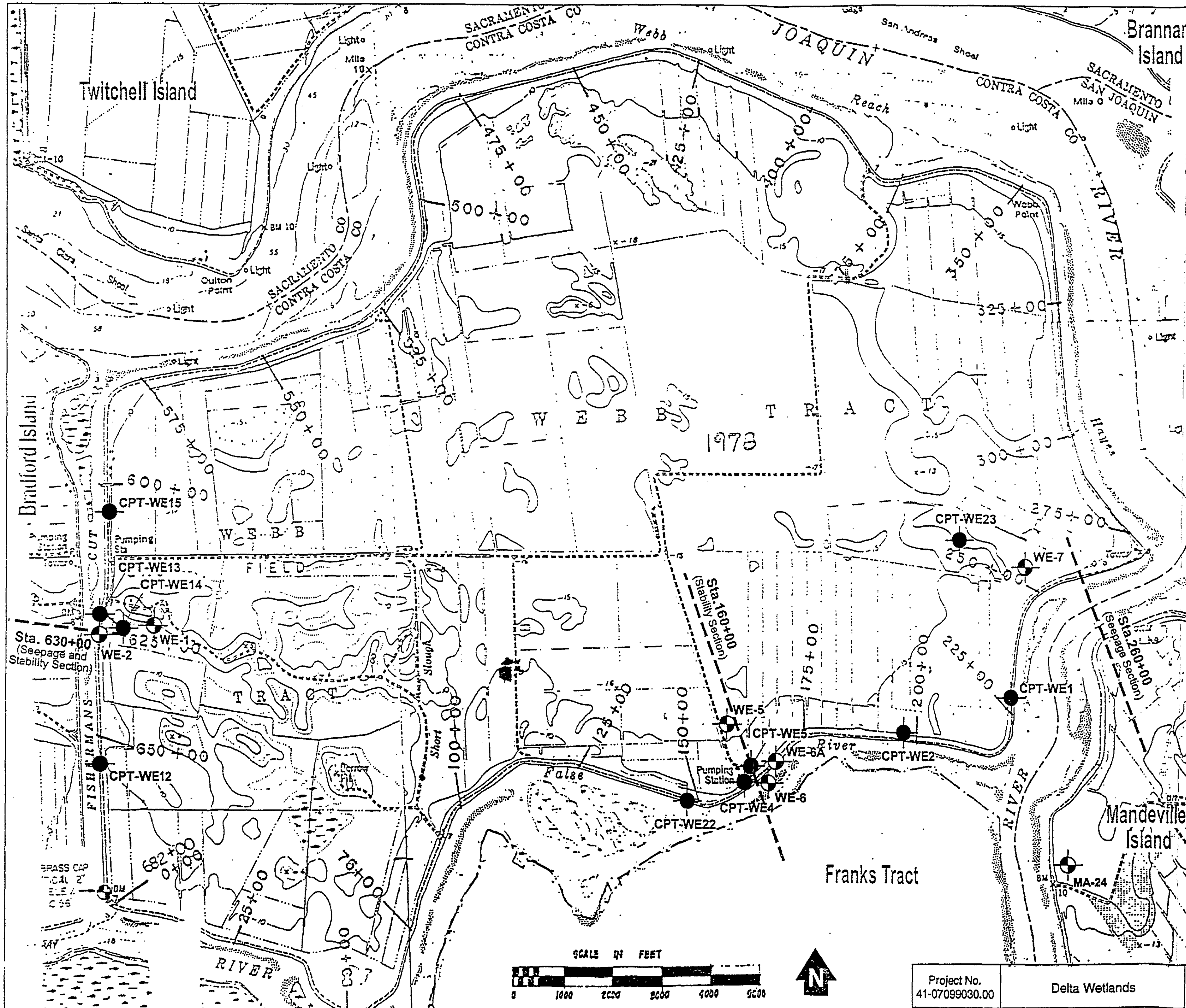
Table 2.3.2 continued

Case	Description	Figure Number	Head in Sand at Near Levee CL (feet)	Flow at Near Levee CL (gpm/ft)	Head in Sand at Far Levee CL (feet)	Flow at Far Levee CL (gpm/ft)	Water Table At Far Toe of Far Levee (feet)	Pumping Rate Required for Wells at 160' (gpm)
BORROW AREA ANALYSIS								
BA1	Webb Tract - Station 630+00 Full Reservoir w/ Pumping	2.3.16	-15	0.0738 (pumping)	-15	0.0071	-16	12
BA2	Webb Tract - Station 630+00 Full Reservoir w/ Pumping (Borrow Area 400' from Levee Toe)	2.3.16	-15	0.0745 (pumping)	-15	0.0071	-16	12



LEGEND

Approximate Borehole Location

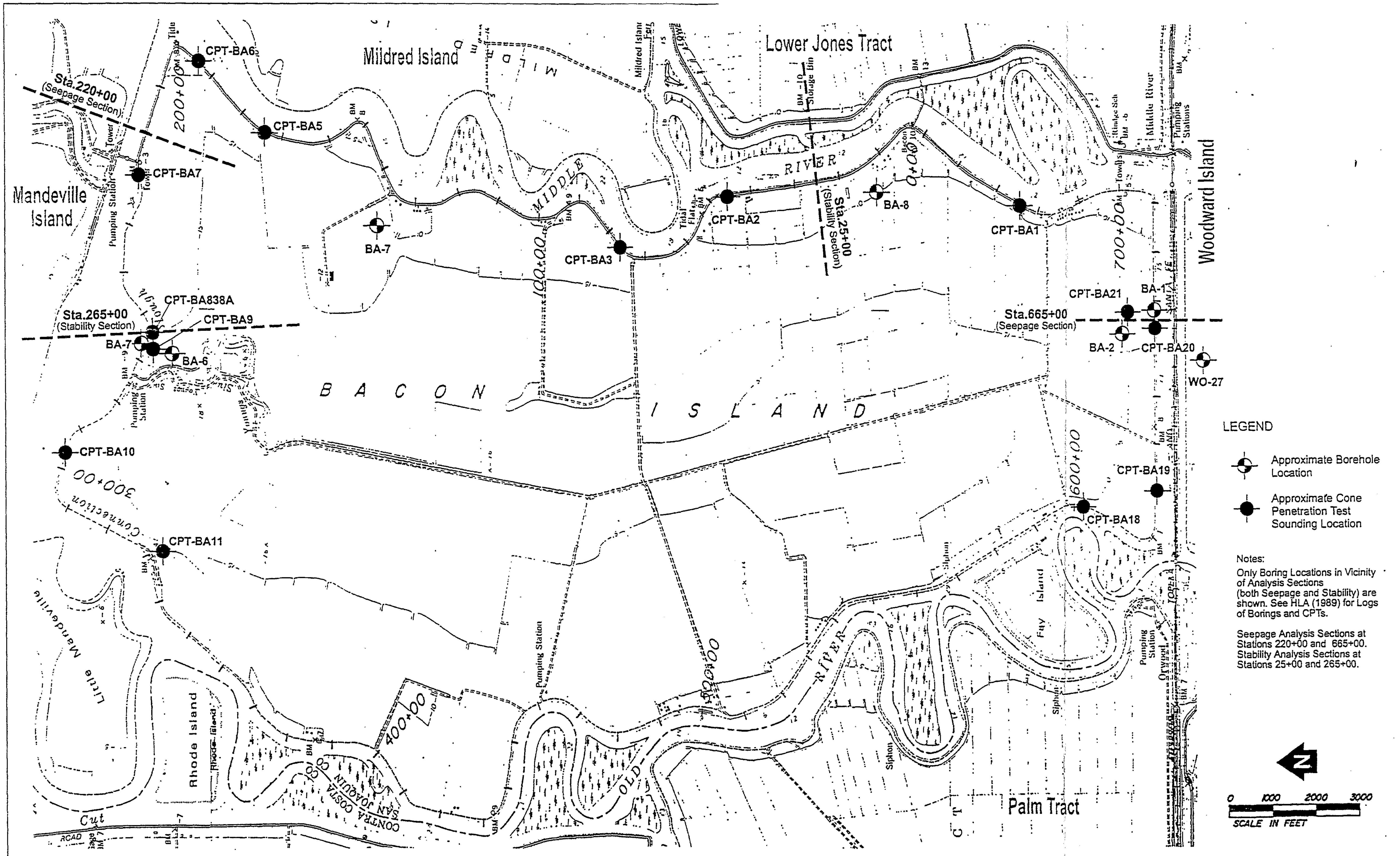
Approximate Cone Penetration Test Sounding Location

Notes: Only Boring Locations in Vicinity of Analysis Sections (both Seepage and Stability) are shown. See HLA (1989) for Logs of Borings and CPTs.

Seepage Analysis Sections at Stations 260+00 and 630+00. Stability Analysis Sections at Stations 160+00 and 630+00.

Project No. 41-07099030.00	Delta Wetlands	WEBB TRACT BASE MAP	Figure 2.2.1
URS Greiner Woodward Clyde			

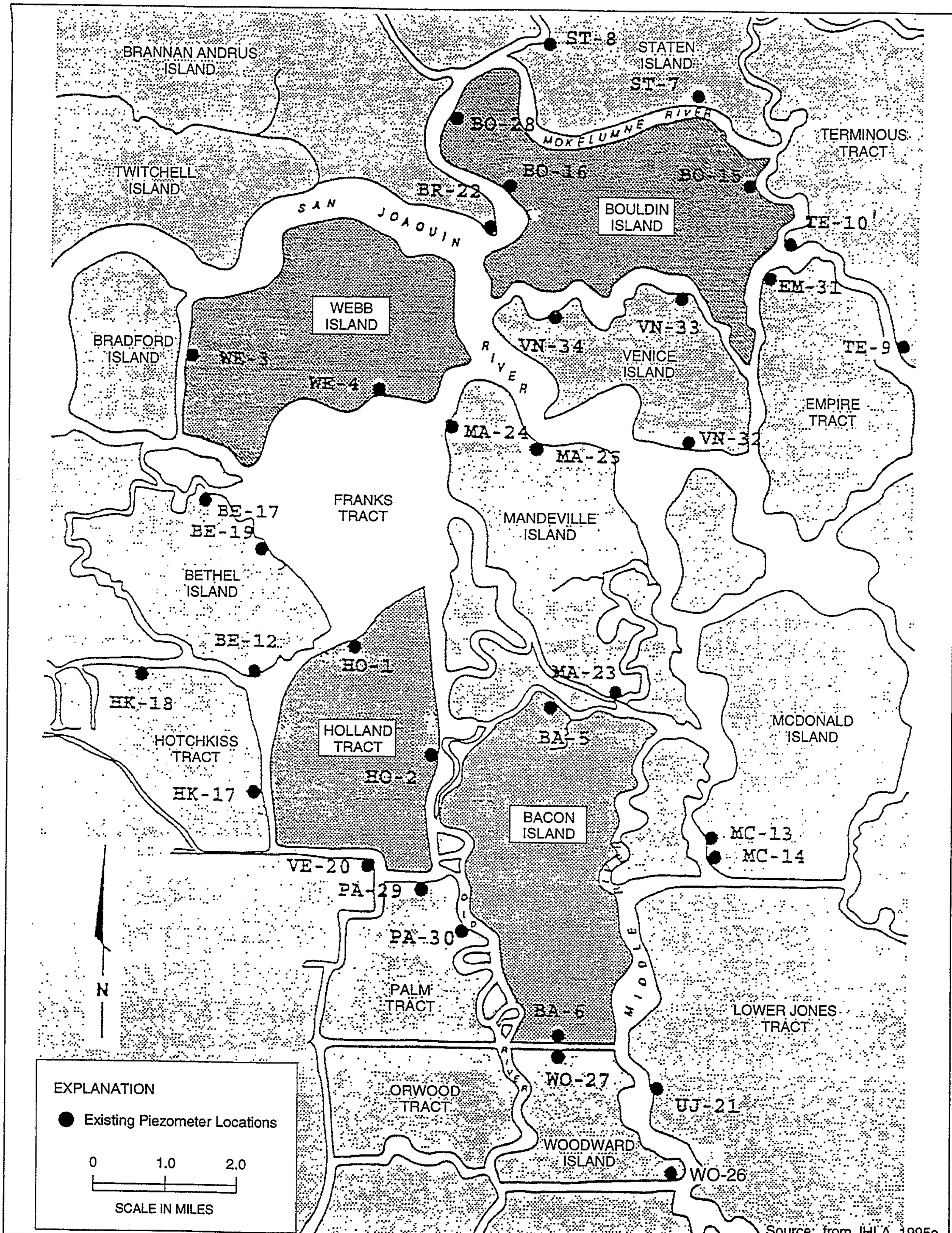
41-07099030.00-00003/120999/gos



Project No. 41-07099030.00	Delta Wetlands	BACON ISLAND BASE MAP	Figure 2.2.2
URS Greiner Woodward Clyde			

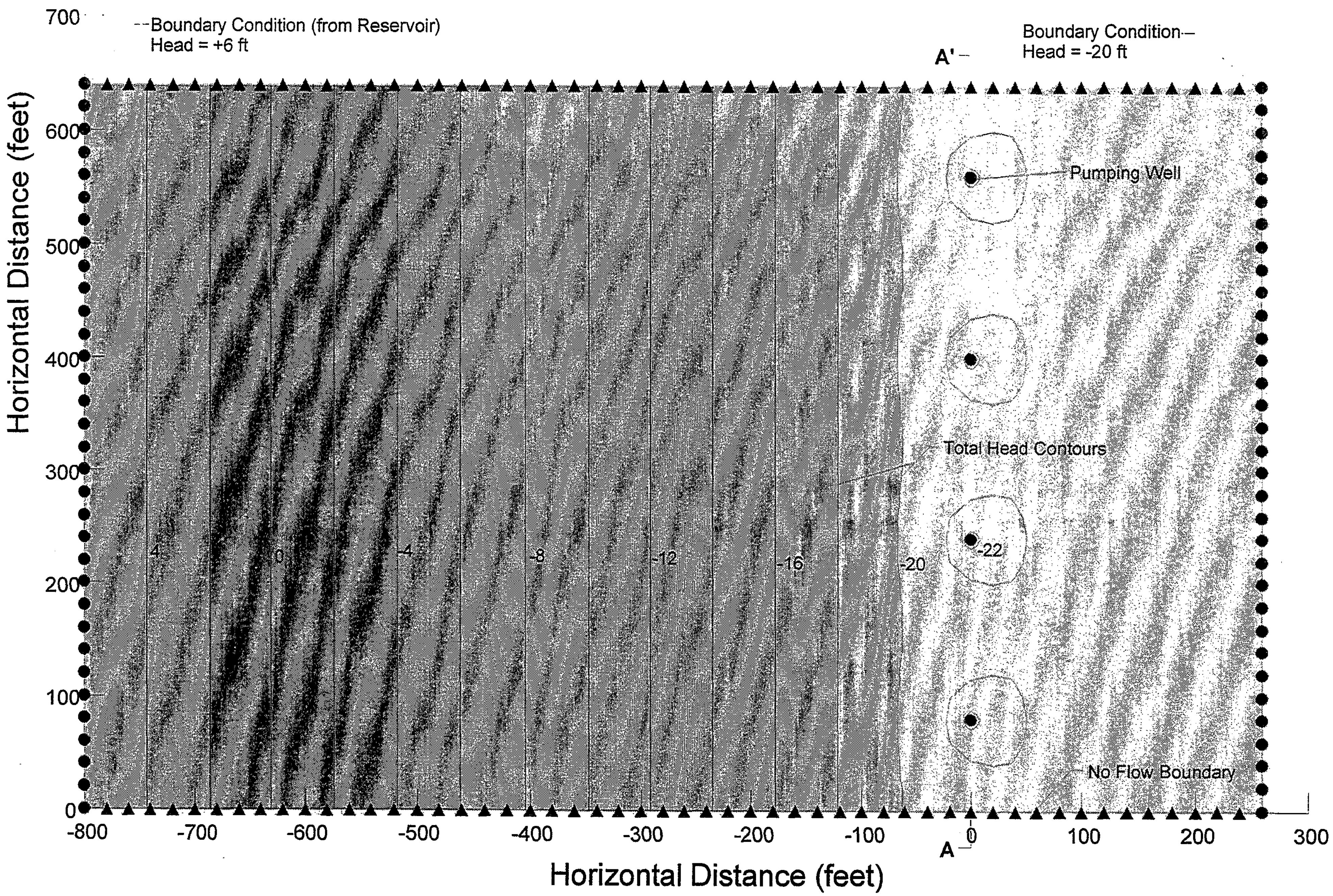
41-07099030.00-00003/120999/gos

C-063418

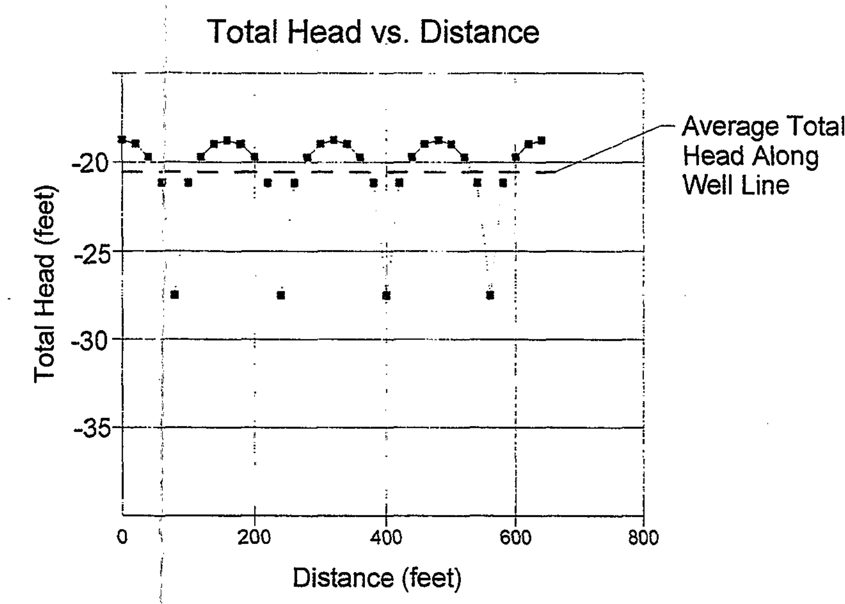


Project No. 41-07099030.00	Delta Wetlands	LOCATIONS OF EXISTING BACKGROUND GROUNDWATER MONITORING WELLS	Figure 2.2.3
URS Greiner Woodward Clyde			

41-07099030.00-00003/051800/gos



EXAMPLE OF PLAN VIEW MODEL APPROACH



TOTAL HEAD ALONG WELL LINE
(SECTION A-A')
160' WELL SPACING

DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 SEEPAGE ANALYSIS APPROACH PLAN VIEW MODEL	FIGURE 2.3.1
URS GREINER WOODWARD CLYDE		

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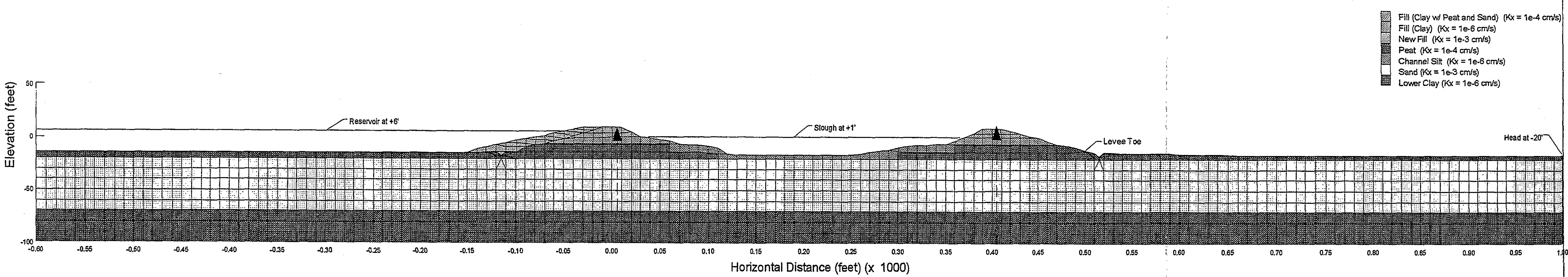
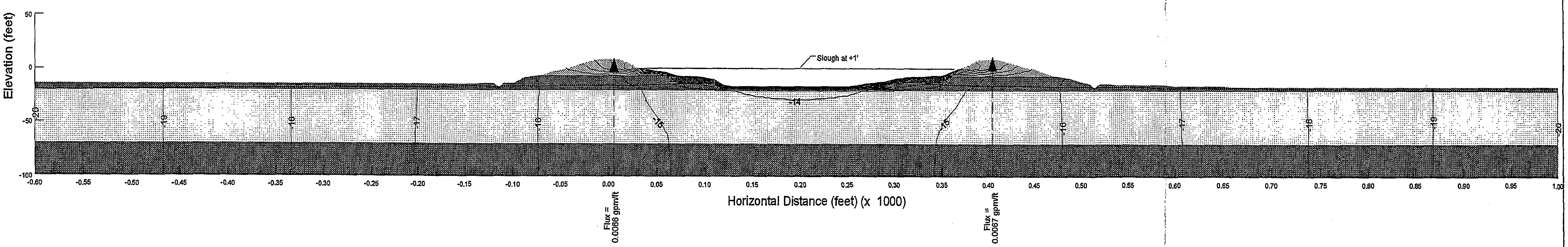


A. Stratigraphy and Model Mesh



B. Existing Seepage Conditions
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 STRATIGRAPHY AND EXISTING SEEPAGE CONDITIONS	FIGURE 2.3.2
URS GREINER WOODWARD CLYDE		

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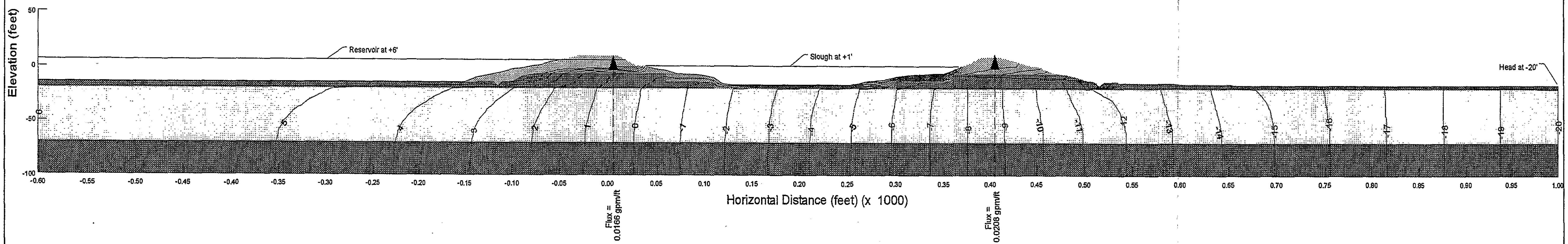
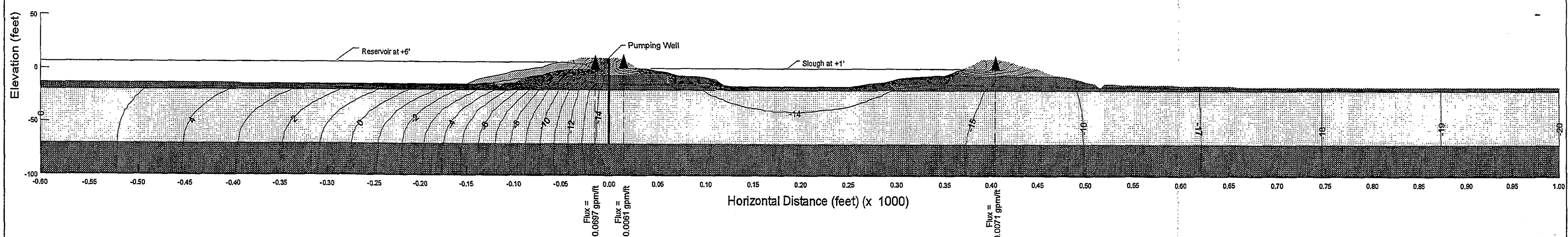


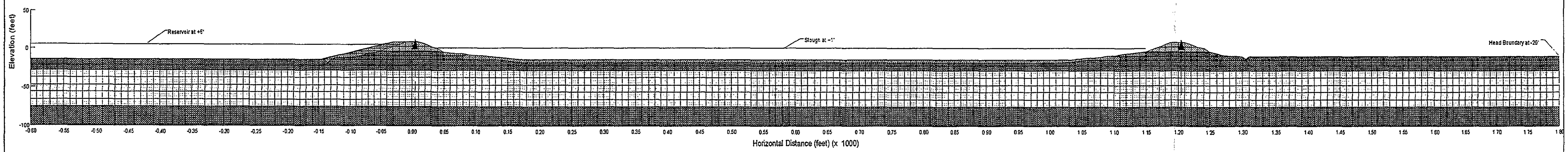
A. Seepage Condition - Full Reservoir, No Pumping
Total Head Contours



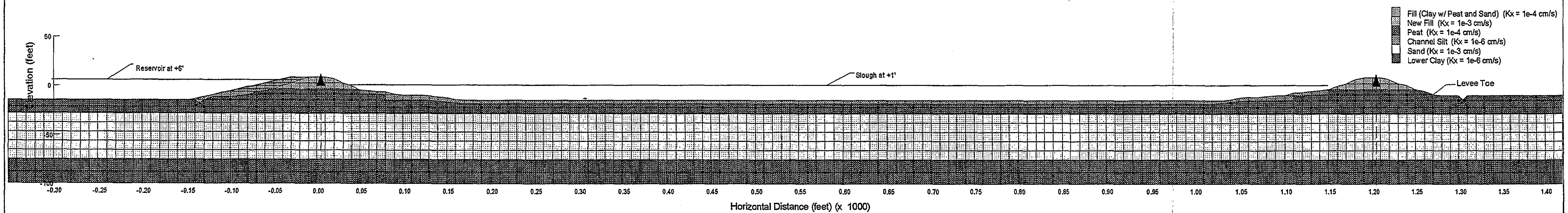
B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR	FIGURE 2.3.3
URS GREINER WOODWARD CLYDE		

C—063422

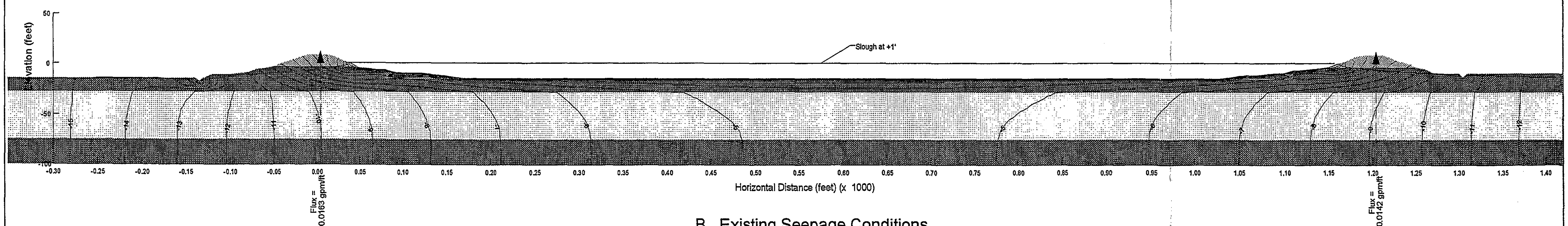


Full Model



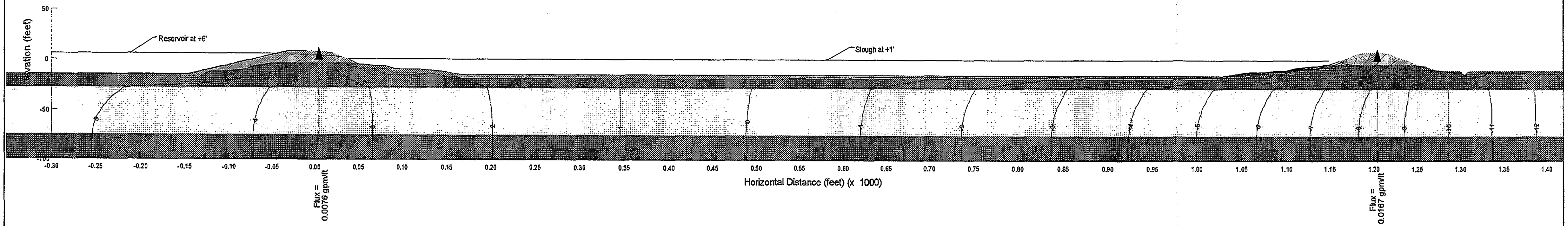
Partial Model

A. Stratigraphy and Model Mesh

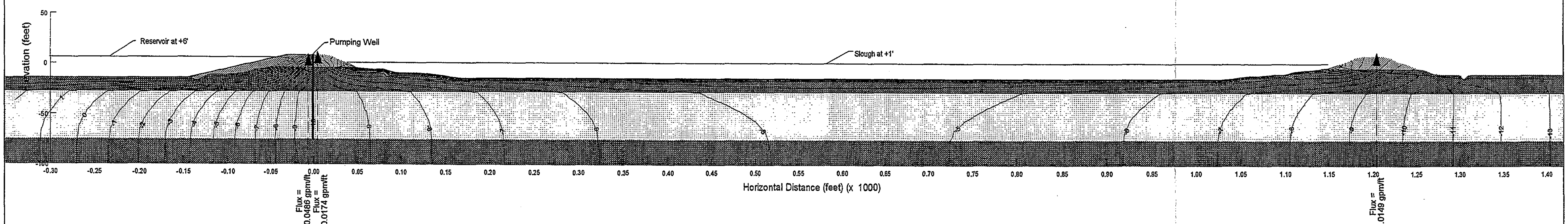


B. Existing Seepage Conditions
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 260+00 STRATIGRAPHY AND EXISTING SEEPAGE CONDITIONS	FIGURE 2.3.4
URS GREINER WOODWARD CLYDE		



A. Seepage Condition - Full Reservoir, No Pumping
Total Head Contours



B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 260+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR	FIGURE 2.3.5
URS GREINER WOODWARD CLYDE		

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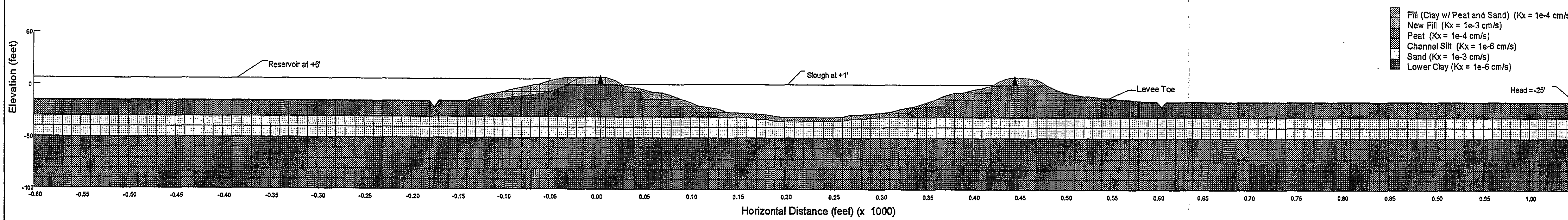
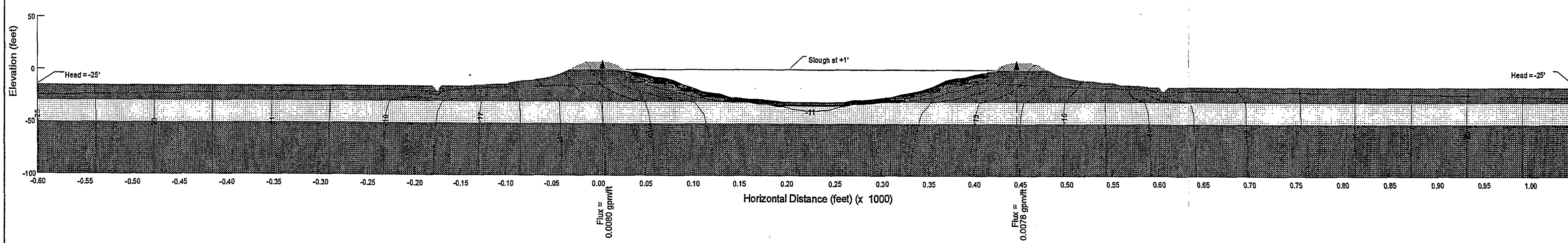
C — 0 6 3 4 2 4

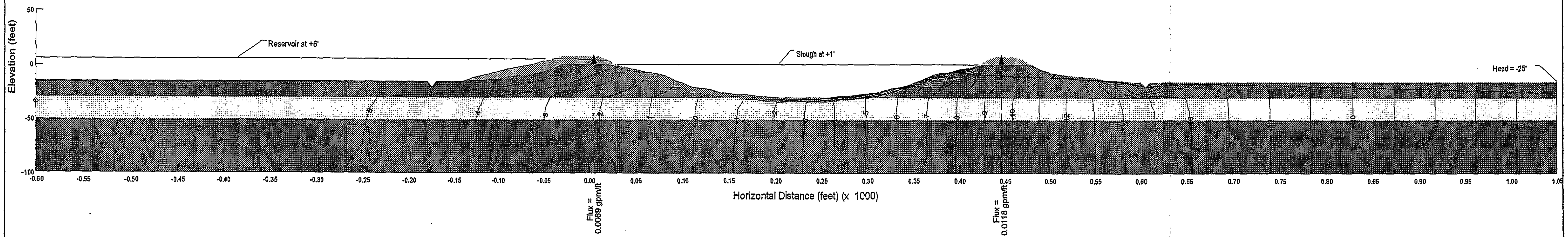


A. Stratigraphy and Model Mesh

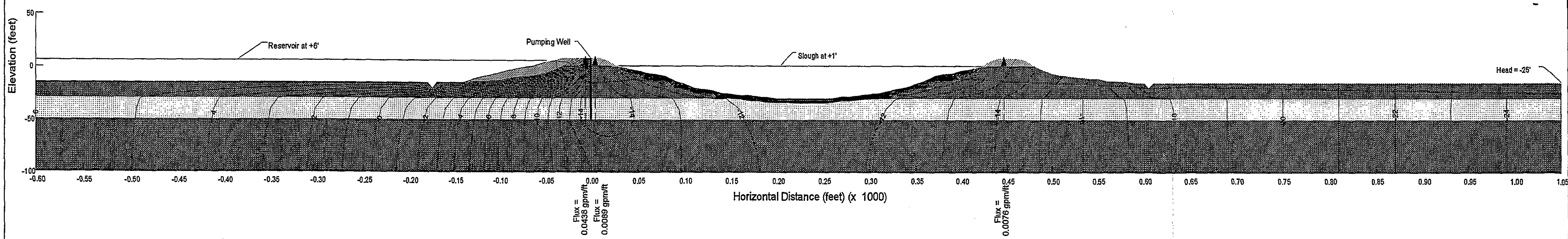


B. Existing Seepage Conditions
Total Head Contours

DELTA WETLANDS PROJECT	BACON ISLAND STATION 220+00 STRATIGRAPHY AND EXISTING SEEPAGE CONDITIONS	FIGURE
URS GREINER WOODWARD CLYDE		2.3.6



A. Seepage Condition - Full Reservoir, No Pumping
Total Head Contours



B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours

DELTA WETLANDS PROJECT	BACON ISLAND STATION 220+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR	FIGURE
URS GREINER WOODWARD CLYDE		2.3.7

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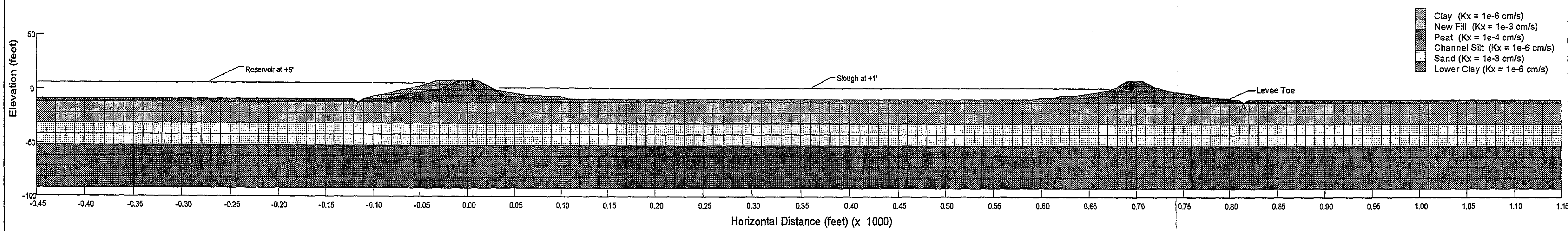
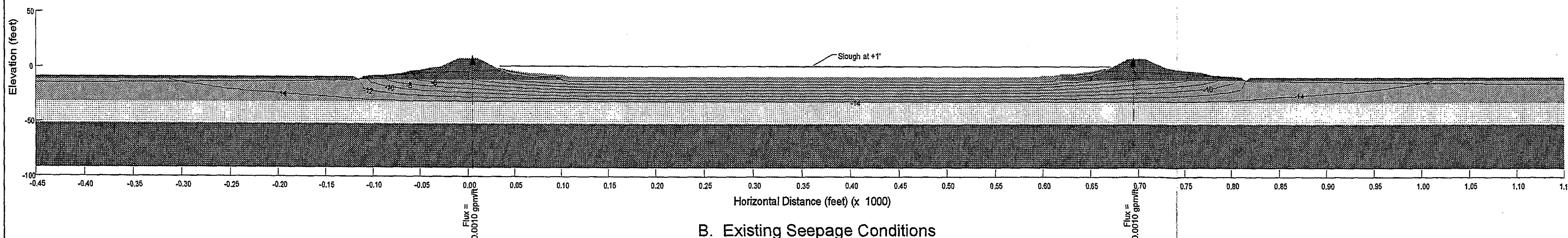
C-063426

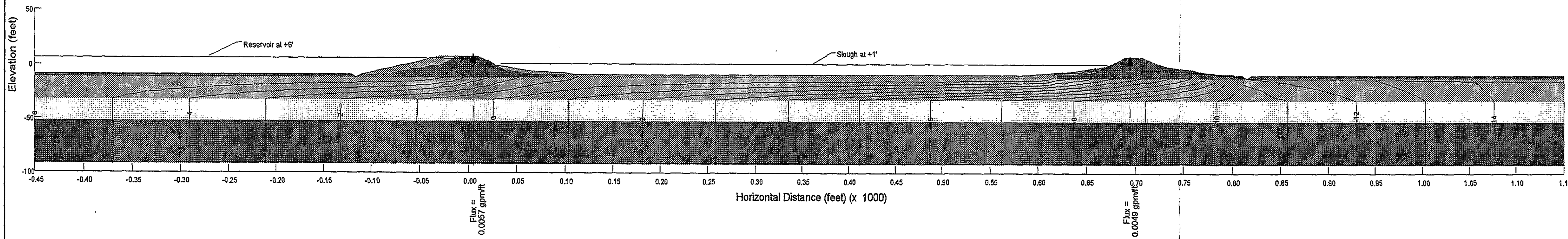


A. Stratigraphy and Model Mesh

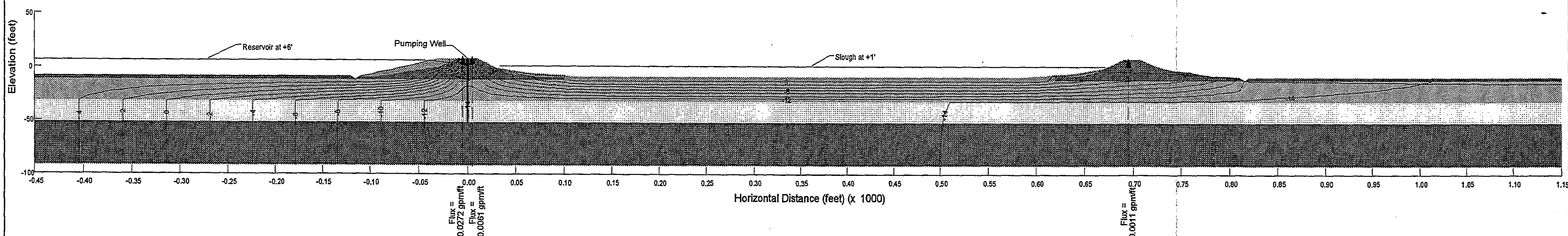


B. Existing Seepage Conditions
Total Head Contours

DELTA WETLANDS PROJECT	**BACON ISLAND STATION 665+00 STRATIGRAPHY AND EXISTING SEEPAGE CONDITIONS**	**FIGURE 2.3.8**
URS GREINER WOODWARD CLYDE		

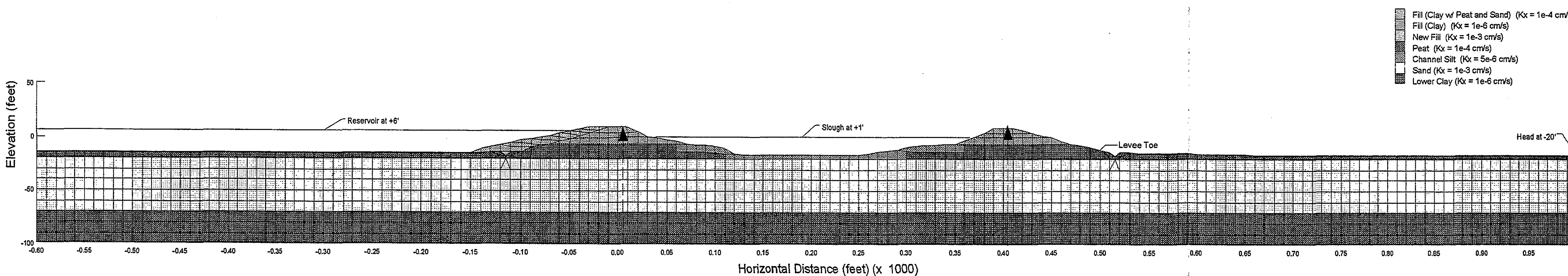


A. Seepage Condition - Full Reservoir, No Pumping
Total Head Contours

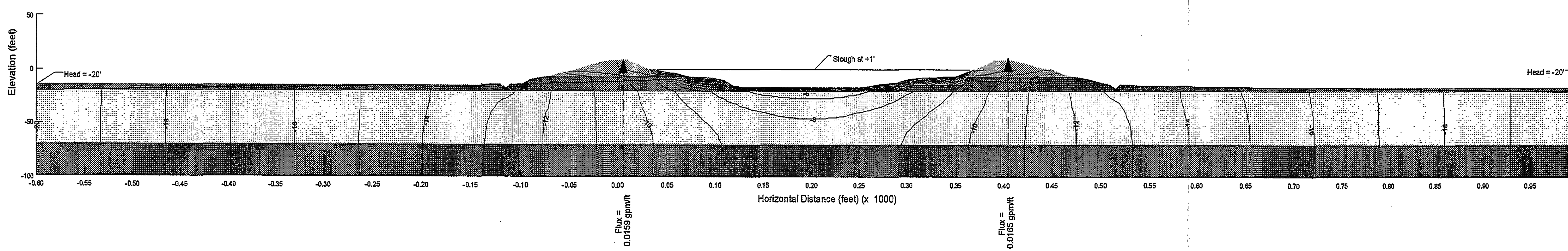


B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours

DELTA WETLANDS PROJECT	**BACON ISLAND STATION 665+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR**	**FIGURE 2.3.9**
URS GREINER WOODWARD CLYDE		

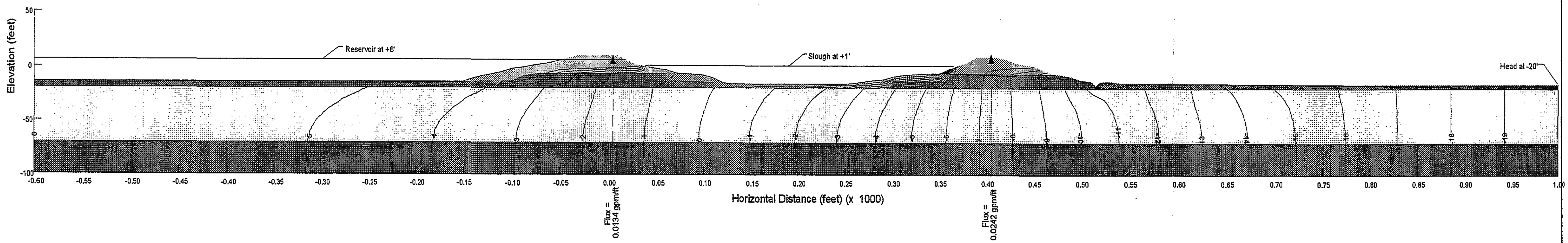


A. Stratigraphy and Model Mesh

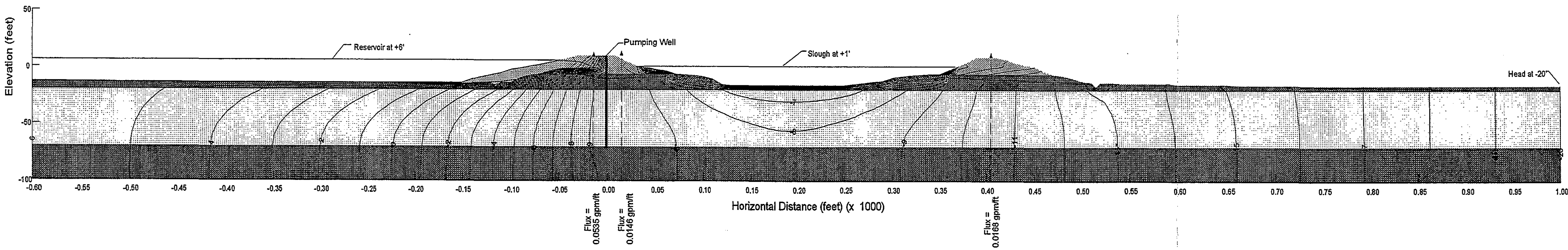


B. Existing Seepage Conditions
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 STRATIGRAPHY AND EXISTING SEEPAGE CONDITIONS (Channel Silt at 5e-6 cm/s)	FIGURE 2.3.10
URS GREINER WOODWARD CLYDE		



A. Seepage Condition - Full Reservoir, No Pumping
Total Head Contours



B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR (Channel Silt at 5e-6 cm/s)	FIGURE 2.3.11
URS GREINER WOODWARD CLYDE		

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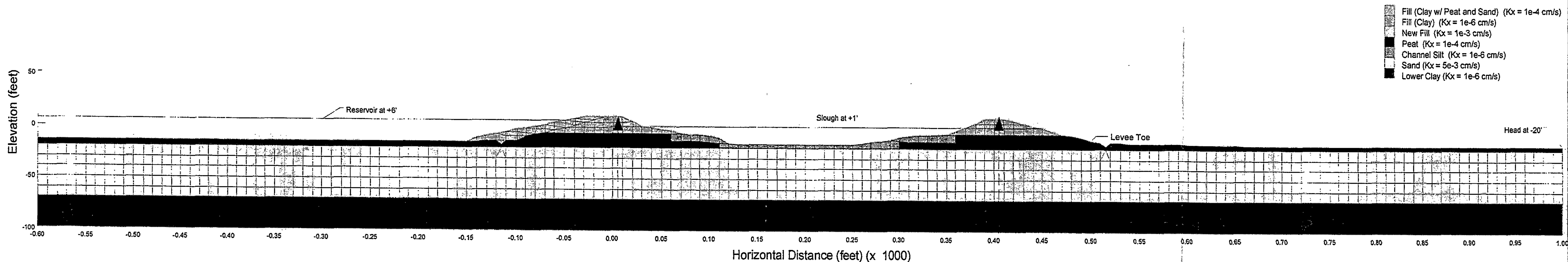
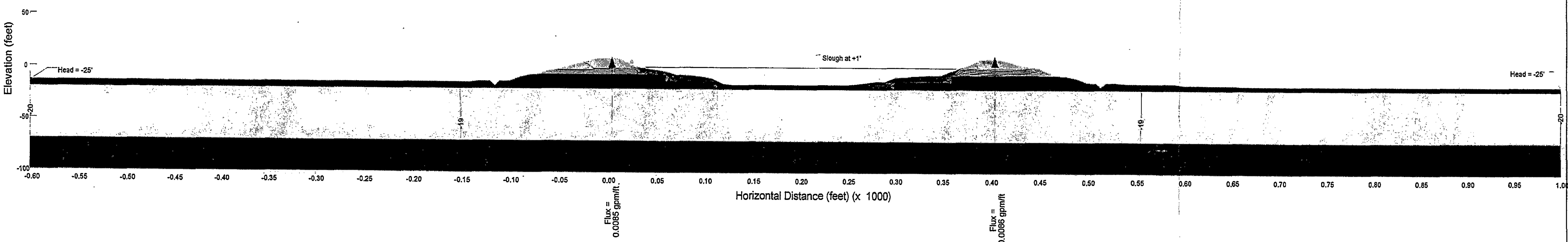


A. Stratigraphy and Model Mesh



B. Existing Seepage Conditions
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 STRATIGRAPHY AND EXISTING SEEPAGE CONDITIONS (Aquifer Sand at 5e-3 cm/s)	FIGURE 2.3.12
URS GREINER WOODWARD CLYDE		

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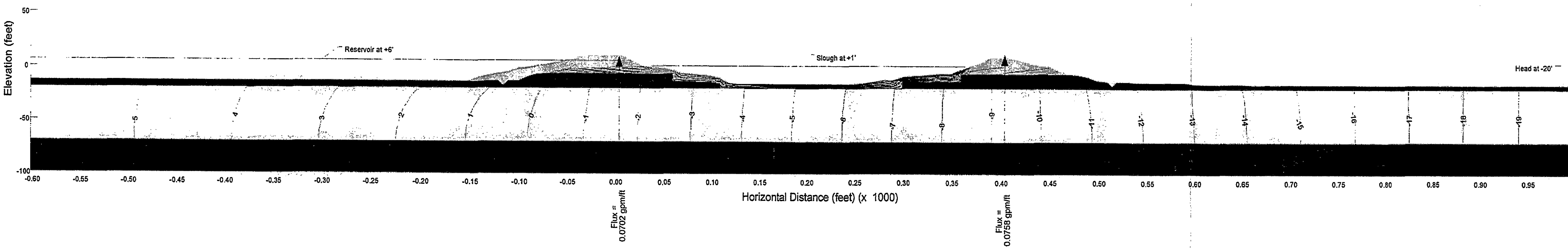
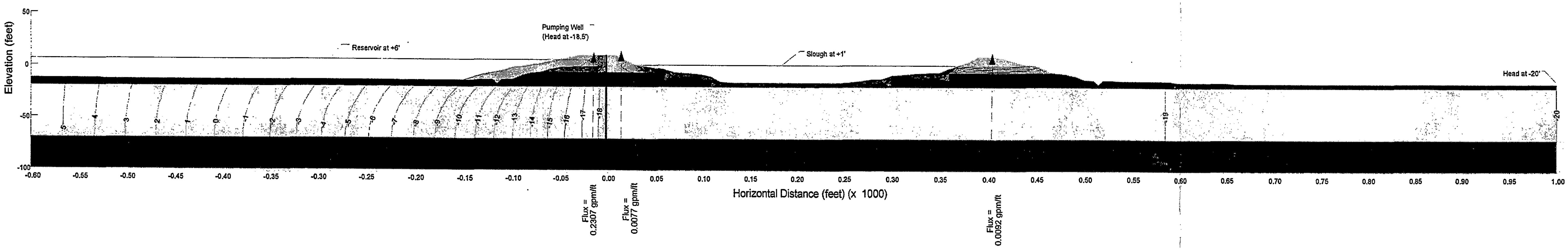
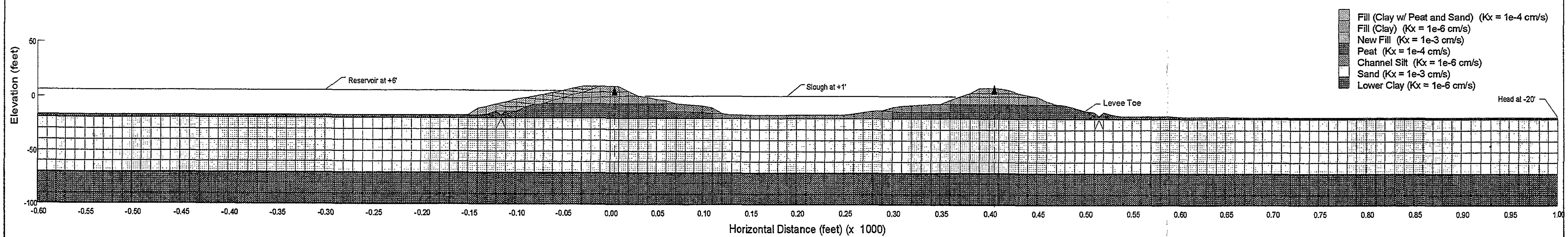


A. Seepage Condition - Full Reservoir, No Pumping
Total Head Contours

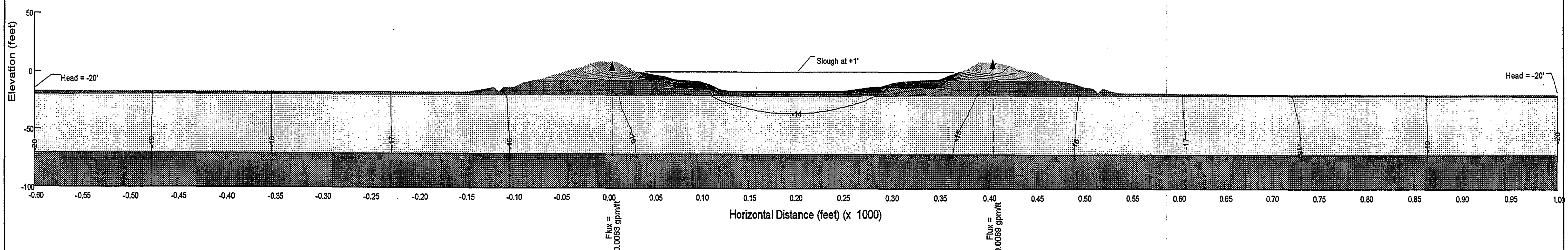


B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	WEBB TRACT STATION 630+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR (Aquifer Sand at 5e-3 cm/s)	FIGURE 2.3.13

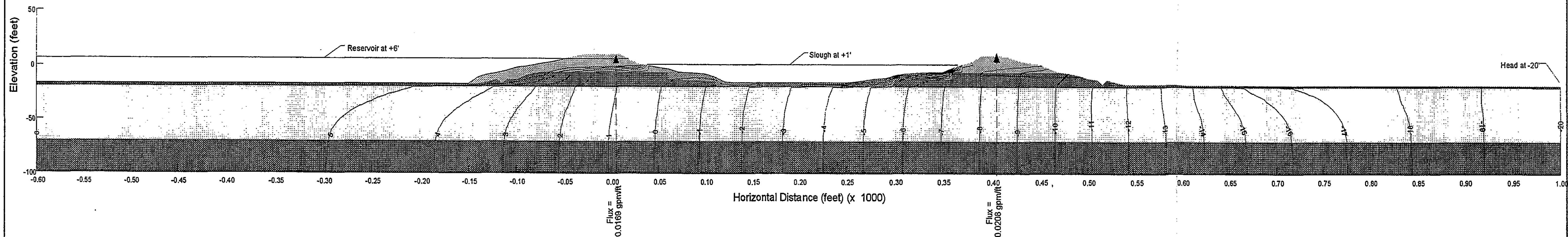


A. Stratigraphy and Model Mesh

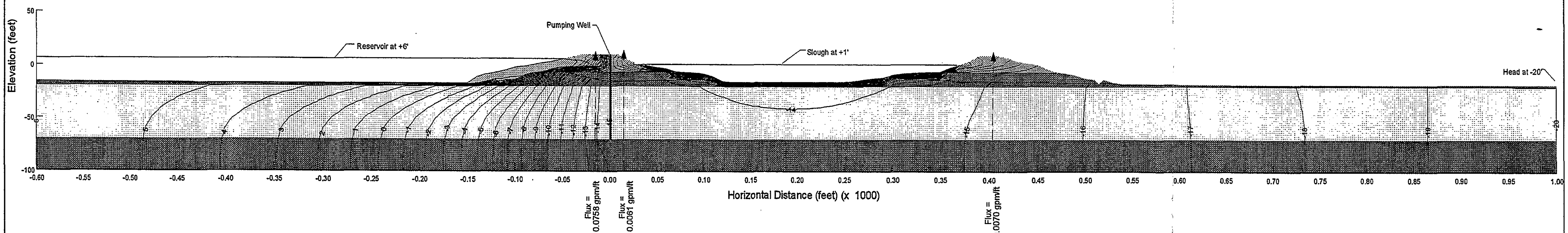


B. Existing Seepage Conditions
Total Head Contours

DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 STRATIGRAPHY AND EXISTING SEEPAGE CONDITIONS (Peat thickness reduced from 6 feet to 3 feet)	FIGURE 2.3.14
URS GREINER WOODWARD CLYDE		

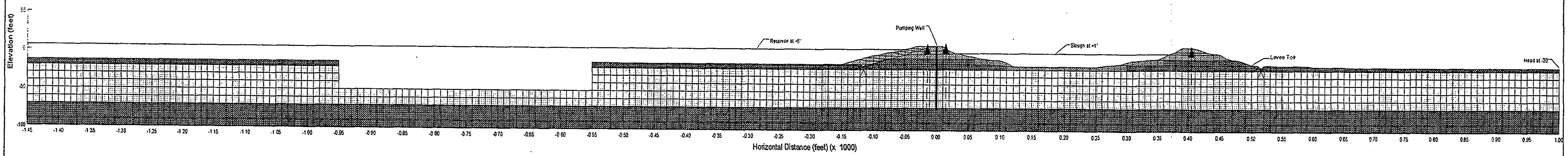


A. Seepage Condition - Full Reservoir, No Pumping
Total Head Contours

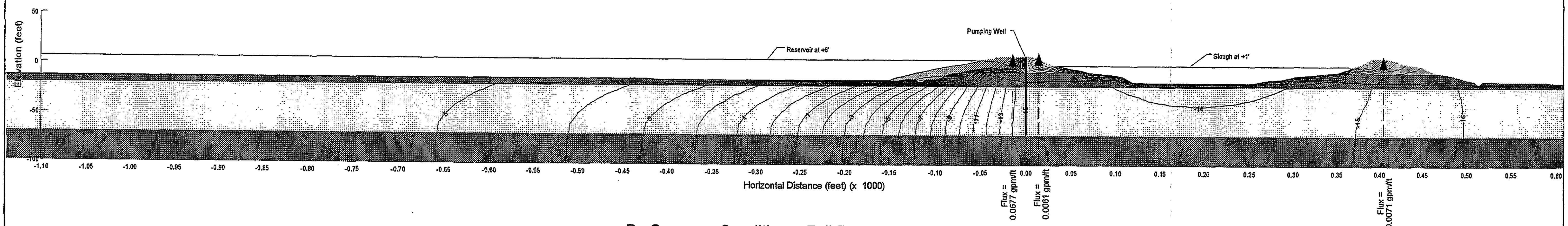


B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours

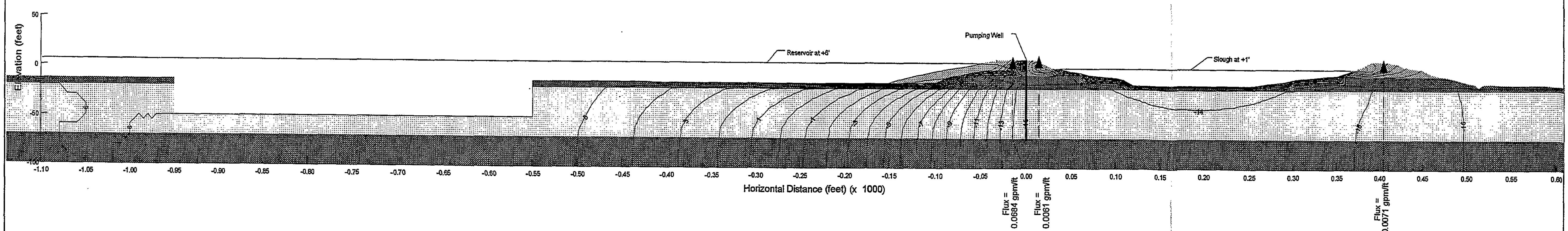
DELTA WETLANDS PROJECT	**WEBB TRACT STATION 630+00** **SEEPAGE CONDITIONS WITH FULL RESERVOIR** **(Peat thickness reduced from 6 feet to 3 feet)**	**FIGURE** **2.3.15**
URS GREINER WOODWARD CLYDE		



A. Stratigraphy and Extended Model Mesh

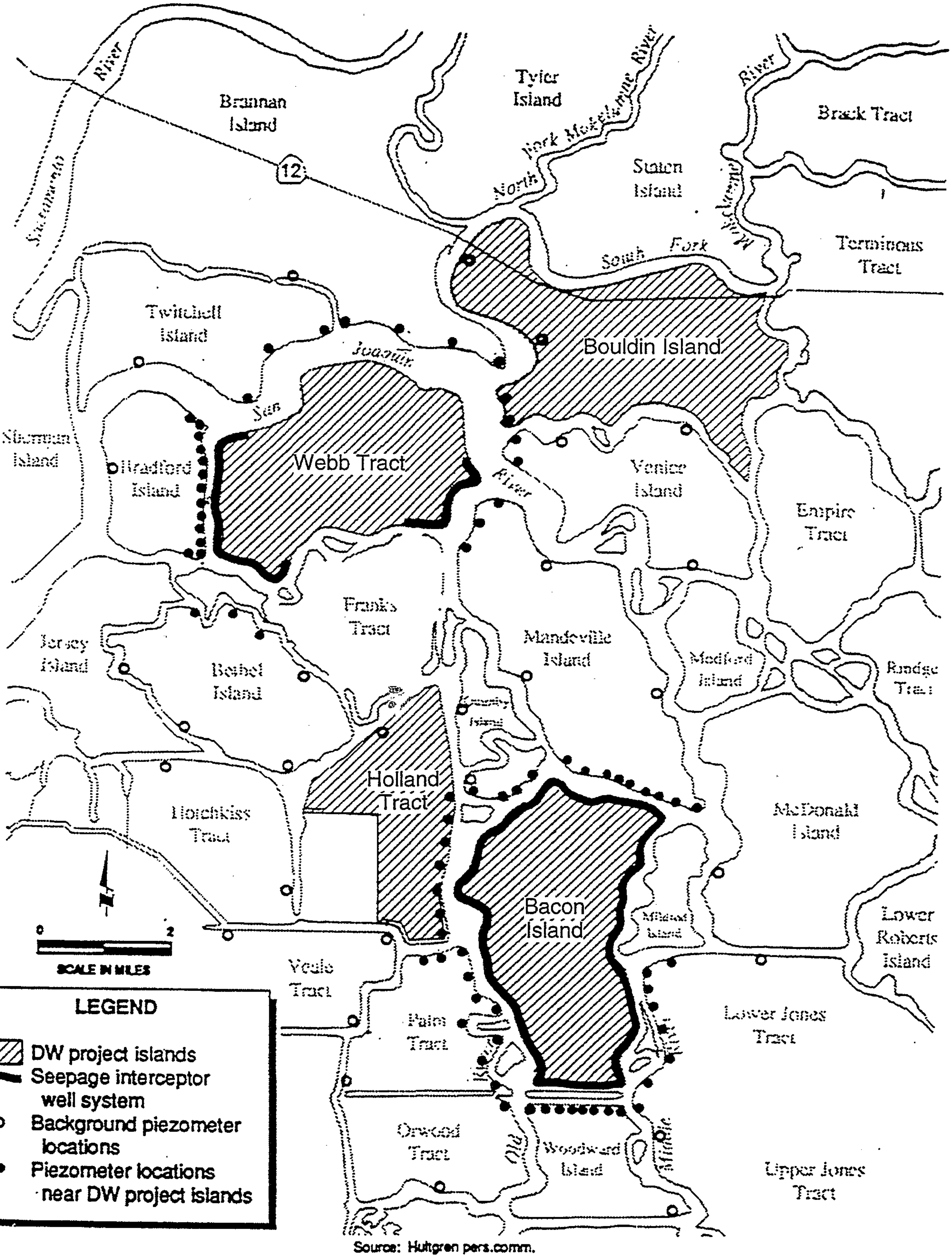


B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours under Existing Conditions



C. Seepage Condition - Full Reservoir with Pumping
Total Head Contours with Borrow Area

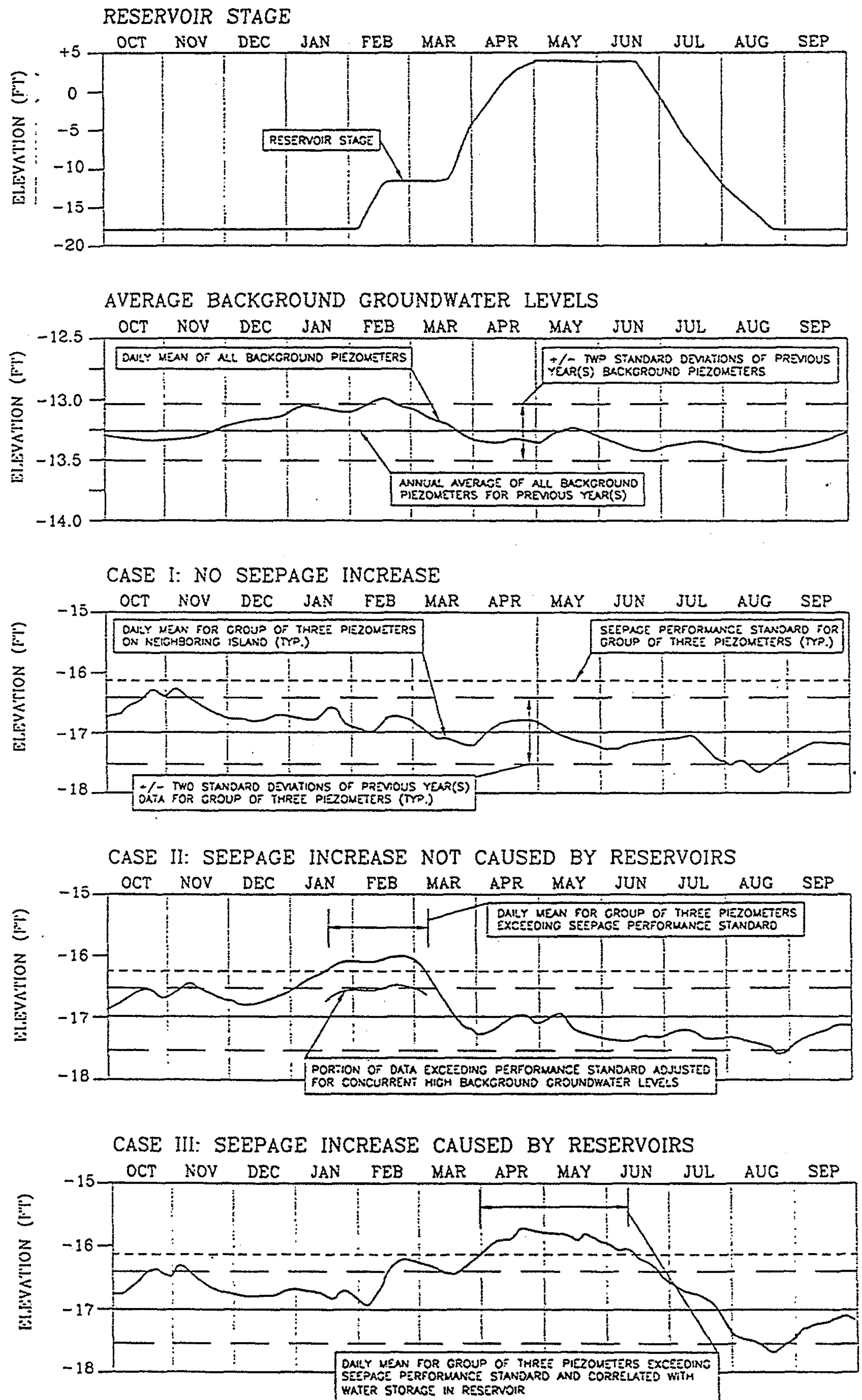
DELTA WETLANDS PROJECT	WEBB TRACT STATION 630+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR (With Effects of Borrow Area)	FIGURE 2.3.16
URS GREINER WOODWARD CLYDE		



Source: from Jones and Stokes, 1995

Project No. 41-07099030.00	Delta Wetlands	PROPOSED LOCATIONS OF SEEPAGE MONITORING PIEZOMETERS	Figure 2.4.1
URS Greiner Woodward Clyde			

41-07099030.00-00003/120999/gos



Source: from Hultgren, 1997a

Project No. 41-07099030.00	Delta Wetlands	HYPOTHETICAL PATTERNS OF SEEPAGE RELATIVE TO PERFORMANCE STANDARDS	Figure 2.4.2
URS Greiner Woodward Clyde			

41-07099030.00-00003/120999/gos

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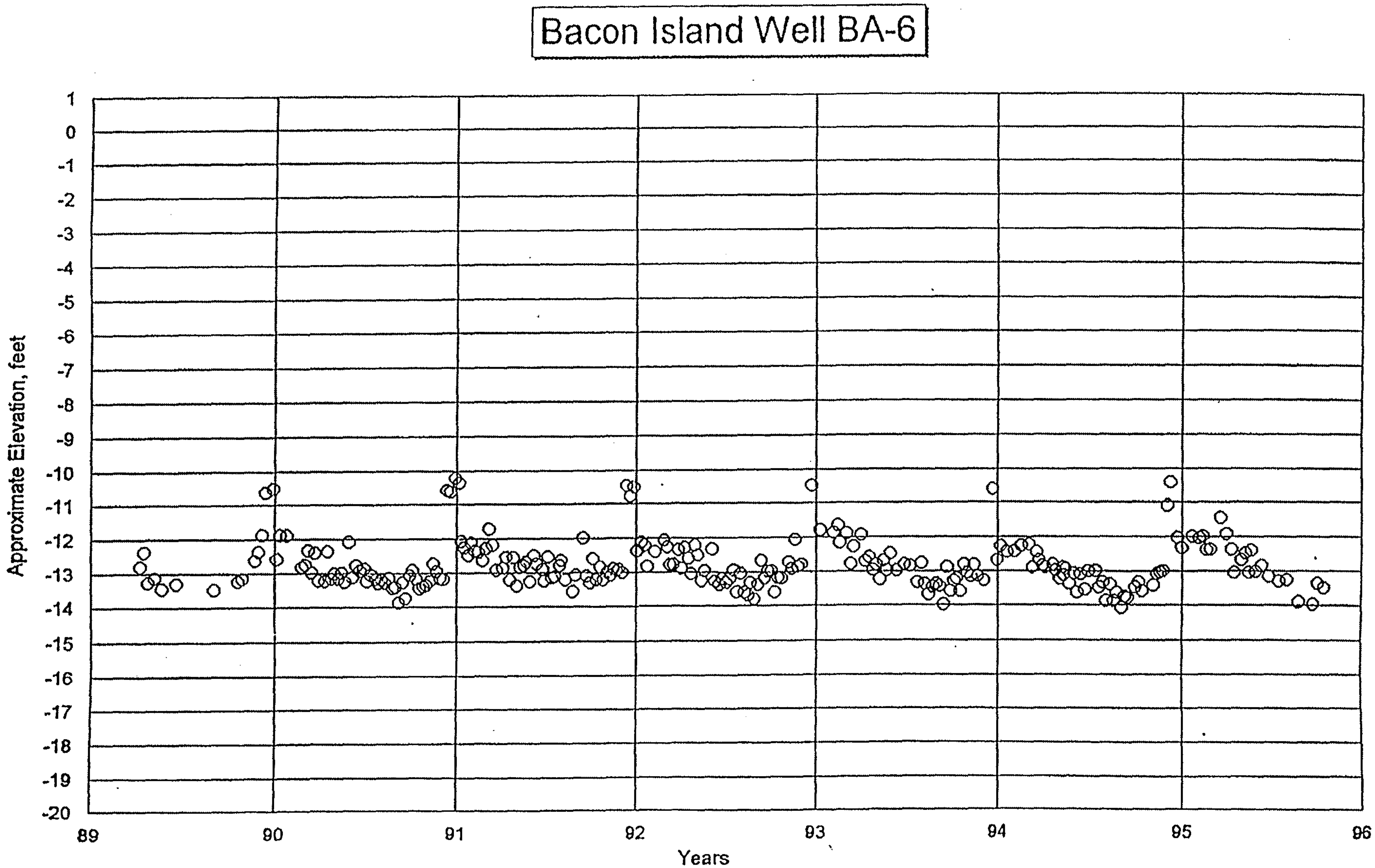

Source: from HLA 1995c

Project No. 41-07099030.00	Delta Wetlands	WATER LEVEL DATA FROM BACKGROUND MONITORING WELL BA-6	Figure 2.4.3
URS Greiner Woodward Clyde			

41-07099030.00-00003/120999/gos

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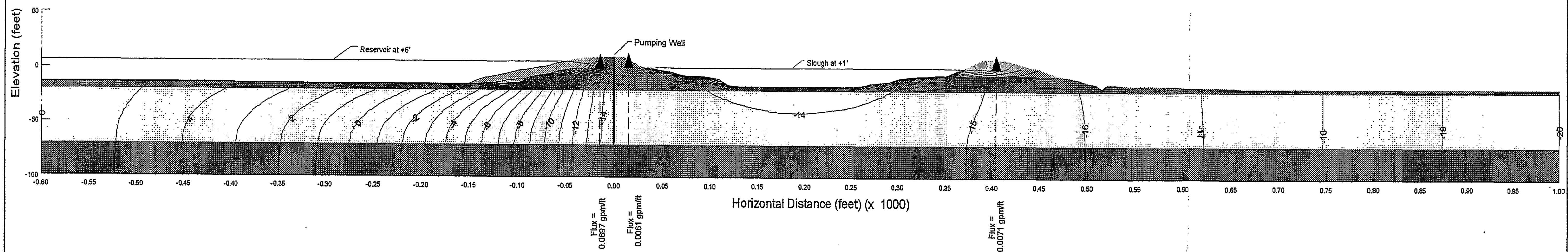
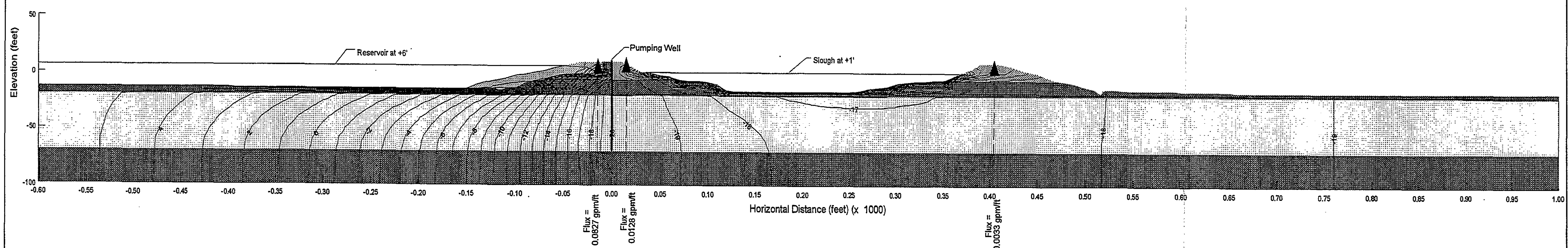


A. Seepage Condition - Full Reservoir with Pumping
Total Head Contours
Head in Pumping Well at -15 ft.



B. Seepage Condition - Full Reservoir with Pumping
Total Head Contours
Head in Pumping Well at -20 ft.

DELTA WETLANDS PROJECT	**WEBB TRACT STATION 630+00 SEEPAGE CONDITIONS WITH FULL RESERVOIR ANALYSIS OF INADVERTENT WATER DIVERSIONS**	**FIGURE** **2.6.1**
URS GREINER WOODWARD CLYDE		

3.1 OBJECTIVES

The main objective of the stability analysis was to evaluate the proposed levee strengthening scheme for Webb Tract and Bacon Island in the Delta Wetlands (DW) Project. In particular, the adequacy of the proposed levee configuration in relation to static and dynamic slope stability of the levee was studied. Additionally, other performance conditions were evaluated including bearing capacity, slope deformations and settlement and their effects on levee stability, and potential effects associated with geologic and seismic hazards (e.g., liquefaction).

As part of this study, we performed the tasks listed below:

- Evaluated analysis results and soil engineering parameters used in previous studies conducted on levee stability;
- Assessed assumptions made related to subsurface soil and groundwater conditions used in slope stability analysis;
- Conducted additional static slope stability analyses for the existing conditions and the proposed strengthened levee configurations for various scenarios including end-of-construction, long-term, sudden drawdown, and seismic performance (quasi-static);
- Reviewed previous and relevant ground motion studies for the project area;
- Developed and updated dynamic soil parameters based on recent findings and published data;
- Developed site-specific design response spectra, and acceleration time histories for additional dynamic analyses;
- Conducted two-dimensional dynamic analyses of the proposed levee configuration and assessed post-seismic deformation;
- Assessed the liquefaction potential at the site, and estimated the potential liquefaction-induced settlement, and its effect on the performance of the proposed levee design; and
- Evaluated the borrow needs for the levee strengthening, including volume estimation and borrow sources.

3.2 REVIEW OF PREVIOUS SLOPE STABILITY ANALYSES

A number of geotechnical studies that include slope stability analyses and other issues related to the overall stability of the Delta levees and their performance have been conducted by Harding Lawson Associates (HLA), Hultgren Geotechnical Engineers and Hultgren Tillis Engineers for the DW Project. We reviewed the geotechnical data, assumptions made, and results from stability analyses contained in those reports. The principal reports in connection with levee slope stability analysis include HLA (1989), HLA (1992e) and HLA (1993a).

HLA (1989) contains results of a preliminary geotechnical investigation conducted by HLA for the DW project. The report describes the subsurface soil conditions encountered during a field investigation that included drilling, standard penetration testing (see Glossary), sampling, and cone penetration test (CPT) sounding. The field work was performed in Webb Tract, Bacon Island and in the neighboring Bouldin Island and Holland Tract. For our slope stability analysis, we relied on the geotechnical data encountered in borings and CPT soundings. The HLA (1989) report also presents engineering soil properties determined in a laboratory testing program conducted on a limited number of samples from the borings. Soil tests included particle size analyses, consolidation tests, moisture content, dry density, shear strength, and permeability tests. The report also contains results of slope stability analyses for the proposed strengthened levee configuration. They analyzed the stability of slopes facing the reservoir islands and the channel and evaluated potential settlements.

HLA (1992e) discusses geotechnical investigations and design for Wikerson Dam on Bouldin Island. The report contains useful geotechnical data.

The HLA (1993a) letter report discusses further issues regarding slope stability of the levee improvements.

The above reports indicate that in the interior of the proposed reservoir islands (Webb Tract and Bacon Island) the subsurface soil conditions consist of a top layer of peat underlain successively by silty sand, stiff clay and silt, and sand. The peat is fibrous, soft, and highly compressible and has a variable thickness ranging from 10 to 20 feet under the levee. The silty sand underlying the peat is dense to very dense and is encountered in a layer 30 feet to 35 feet thick below Webb Tract and 20 feet to 25 feet thick below Bacon Island. The levees typically are built of an approximately 10-foot-thick layer of sandy to clayey fill at the top, which overlies a mixture of clayey peat and peat fill that overlies the naturally occurring peat layer. Because the levee surface has been subsiding over decades, the levees have been raised periodically. The natural peat layer is underlain by a thick sand layer, which itself is underlain by a clay layer.

3.3 ANALYSIS PARAMETERS

We performed independent slope stability analyses to assess the stability and adequacy of the proposed levee strengthening scheme at four cross sections, two for each island being considered as reservoir islands. Details regarding the loading conditions, the ground topography at the selected sections, the selection of material parameters, ground water levels on slough and reservoir sides, and the types of analyses performed are described below.

3.3.1 Cases Considered For Slope Stability Analysis

Because critical conditions may arise on the slopes facing the channel (slough) side as well as the reservoir island side, the margins of safety against instability for both slopes were assessed. The following analysis conditions were considered for each slope.

a) Existing Conditions

For this case, we considered the present levee, island and channel geometry without stored water. Water levels on the island and slough sides were selected to produce a representatively critical, though not the most critical case. (For instance, the highest water stage is taken at +6 feet, even though maximum flood stages may be somewhat higher for short periods.) The specific water elevations used are shown in the table on page 3-3 and discussed for each case.

b) End-of-Construction

The end-of-construction scenario is the condition occurring immediately after placement of the new fill on the reservoir island side of the levee, as illustrated in Figure 3.3.1 for one case. The fill will be placed in compacted layers. Immediately after the placement of the fill, fine-grained soils such as peat in the levee and foundation will not have had sufficient time to drain the construction-induced excess pore pressures, and consequently will not develop higher shear strengths due to the added surcharge. As a result, at the end of construction, pre-construction undrained shear strengths are used to characterize the cohesive soils of the levee and foundation. Water levels on the island and slough sides were selected to produce a critical case; see Section 3.3.3.

If placement of the new fill is done in several stages, as is typically the case for fills on soft foundation soils, the stability should be evaluated after the application of each stage, to ensure adequate calculated stability for each stage. These calculations, together with field monitoring of fill and foundation performance, would allow safe stage levels and consolidation periods between stages to be selected. The stability analysis for the end-of-construction using multiple stages was not calculated in this report, because this type of construction requires to be detailed in the final design.

c) Long-Term

The analysis of long-term levee stability involves the post construction conditions when strength gain has occurred and normal operations of the reservoir are in place. Water levels on the island and slough sides were selected to produce a critical case.

d) Sudden Drawdown

The sudden drawdown case is the condition occurring on the reservoir island slope when the level of the stored water drops rapidly (e.g., reservoir drawdown during an emergency). Because this drop can occur at a relatively rapid rate, the peat and other fine-grained soils do not have enough time to drain, and undrained strengths are used.

e) Pseudo-Static (Determination of Yield Acceleration)

The stability of slopes during seismic loading is analyzed to determine the yield accelerations. The use of the calculated yield acceleration to estimate earthquake-induced deformation of the levees systems is discussed in Section 3.6. Water levels on the island and slough sides were selected to produce critical cases. However, for the seismic condition toward the island, the water table in the slough was taken at the average elevation of +2 feet; it is customary to assume that the highest flood and the design earthquake do not occur simultaneously.

3.3.2 Sections Selected for Analysis

The criteria used in selecting the most critical analysis sections were the highest elevation differential between the crest and the island or slough side toe and the soil conditions affecting stability results. Based on these criteria, four representative cross sections, two from each island being considered for water impoundment, were selected for stability analysis. The locations of these sections are shown in Figure 2.2.1. The section at Webb Tract Sta. 630+00 crosses Fishermans Cut toward Bradford Island. The section at Webb Tract Sta. 160+00 crosses False River toward Franks Tract. The levee geometry and stratigraphy of the sections at stations 160+00 and 360+00 are depicted in Figures 3.3.1 and 3.3.2, respectively.

The section at Bacon Island Sta. 25+00 crosses Middle River toward Lower Jones Tract. The section at Bacon Island Sta. 265+00 crosses Connection Slough toward Mandeville Island. The levee geometry and stratigraphy of the sections at stations 25+00 and 265+00 are depicted in Figures 3.3.3 and 3.3.4, respectively.

Each section is representative of a reach of levee with similar geometry, levee, and foundation materials. Subsurface conditions were described in the HLA (1989) report. The levee materials generally consist of dredged sand, silt, clay, and peat. The thickness of this fill typically varies between 6 to 10 feet. Beneath the levee is a thick layer of peat down to approximately elevation -30 feet. The thickness of the peat layer varies typically between 15 and 35 feet in these two islands. Underlying the peat is a thick layer of dense sand, below which is typically a stiff clay or dense silt. A typical present condition on the islands is a 20-foot wide crest at approximately elevation +8.5 (all elevations in NGVD), with a 2:1 (H:V) slough side slope and a 4:1 reservoir side slope. It was judged that these four cross sections were representative of the more severe slope stability situations of the levees of both reservoir islands.

Two configurations for the planned new fills were proposed by HLA.

(a) The first configuration consists of a uniform reservoir side slope inclined at about 5:1 from the levee crest to toe.

(b) The second configuration consists of an interior slope at about 3:1 from the levee crest down to near elevation –3 feet and then flattening to a 10:1 slope extending toward the island interior.

Both configurations involved raising the levee crest to about elevation +9 feet and widening it to about 35 feet. This wide levee crest would allow future levee raises without widening the levee. The first configuration of levee strengthening was considered for each analysis section. In addition, the second configuration was considered for one section on Webb Tract only.

3.3.3 Water Table Elevations

At each section and case analyzed, we used reservoir island and slough side water levels that would produce critical cases. For the analysis of the existing condition of the slope toward the island, we considered the water level in the slough to be at flood elevation of +6 feet. The maximum peak flood elevation corresponding to a 100-year flood condition is +7.5 feet. After inspection of a number of gauge recordings and historical data at this site, we noted that the maximum peak flood occurs over a very short period of time and hence will not lead to a steady state condition. Therefore, we considered that a flood elevation of +6 feet was a reasonable and conservative representation of the high stage during the flood event. In the case of the stability of the slope facing the slough, the water level in the slough was considered to be at low tide (i.e., elevation 0 feet). Again, elevation 0 was a reasonable and conservative condition, though not the most conservative possible but rarely occurring short-term condition. In both these existing conditions, the water level in the reservoir island was assumed to be at about 2 feet below the existing ground surface.

We assumed that water would be stored up to elevation +6 feet on both reservoir islands. For the analysis of the end-of-construction toward the island slope, we considered the water level in the slough to be at flood elevation of +6 feet. For Webb Tract Sta. 630+00, two different water levels in the slough were considered for comparison purposes. They were elevations +6 feet and +2 feet, corresponding to flood stage and high tide, respectively. Normally, construction takes place in summer and water level in the slough would be unlikely to reach the flood stage. However, late fall construction and early winter precipitation could cause a condition of little consolidation before a flood stage. The various water levels considered on the island and slough sides are summarized in the table below for different analysis conditions.

WATER ELEVATIONS USED IN SLOPE STABILITY ANALYSES

Condition	Water Elevation (ft)		Side Slope Considered for Analysis
	Slough	Island	
Existing	+6 0	-16 -16	Island Slough
End of Construction	+2 and +6	-16	Island
Long-term Condition	+6 0	-14 +6	Island Slough
Seismic, K_y	+2 0	-14 +6	Island Slough
Sudden Drawdown	0	+6 to -14	Island

3.3.4 Soil Parameters

The HLA (1989) report presents geotechnical data obtained from the field exploration and laboratory testing programs in the Delta Wetlands islands. These data were the main source used to derive soil parameters for the slope stability analyses. To further validate the selected material properties used in the analysis, we reviewed published literature regarding the geotechnical properties of peat (e.g., Marachi et al. 1983 and Dhowian et al. 1980). The sands and sandy silts, which are free draining materials, were represented by their effective strength envelopes, consisting mainly of the effective friction angle obtained from correlation with SPT blow counts (Lambe and Whitman, 1969). A summary of the generalized soil strengths used in the various analyses is presented in the table below.

The HLA (1992e) report presents the results of geotechnical investigation and design studies conducted for Wilkerson Dam on Bouldin Island. The report presents an undrained shear strength envelope for peat based on data collected mainly from undrained triaxial shear tests on soil samples acquired from Bouldin Island. We used this envelope to calculate the variation of undrained shear strength of peat soils as a function of effective consolidation pressure.

SOIL PARAMETERS USED IN SLOPE STABILITY ANALYSES

Material	Total (wet) Unit Weight, γ_s (pcf)	Effective Friction Angle, Φ' (degrees)	Effective Cohesion Intercept, c' (psf)	Undrained Friction Angle, Φ^t (degrees)	Undrained Cohesion Intercept, c (psf)
Existing clay fill with peat and sand	110	27	80	12	135
Existing sand fill	110	32	0	-	-
Existing silty sand fill with fat clay	110	32	0	-	-
Planned fill	120	34	0	-	-
Clay with peat	80	28	100	12	135
Peat under levee	70	28	50	12	135
Free-field peat	70	26	50	12	135
Deep sand	125	36	0	-	-

3.4 METHODS OF STATIC SLOPE STABILITY ANALYSIS

The stability of the levees was analyzed using the limit equilibrium method based on Spencer's procedure of slices as coded in the computer program UTEXAS3 (Wright, 1990). In Spencer's procedure all side forces acting on a slice interface are assumed to have the same inclination and all requirements for static equilibrium are satisfied. The trial-and-error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium conditions are satisfied. UTEXAS3 can be used to compute factors of safety using either circular or general shaped, noncircular shear surfaces.

UTEXAS3 is capable of performing two-stage and three-stage computations to simulate rapid undrained loading following a period of consolidation of the soil. The end-of-construction case was analyzed using both the two-stage procedure and an undrained-strength (Su) analysis. The two-stage procedure requires the input of both the effective strength (S-envelope) and total strength (R-envelope) envelopes for the cohesive materials, such as peat. The undrained-strength analysis used undrained shear strengths based on the HLA strength envelope discussed in Section 3.3.4. The sudden drawdown cases were analyzed using the three-stage method as described in the user's manual for UTEXAS3 by Duncan et al. (1990). The three-stage method requires the input of both the effective and total strength envelopes for the peat and the effective strength envelope for the sand.

The presence of rip-rap on both faces of the levee was not considered in the slope stability analyses, because the rip-rap represents only a small portion of the levee mass. If anything, the rip-rap will strengthen the levee.

3.5 RESULTS OF STATIC SLOPE STABILITY ANALYSIS

3.5.1 Webb Tract

The soil parameters used in the analysis are presented in Section 3.3.4 of this report. Table 3.5.1 summarizes the results of the slope stability analyses for station 630+00 using the first configuration of the new fill; i.e., uniform 5:1 slope. Cross sections showing various conditions along with the potential failure surfaces toward the island and the slough are shown in Figures 3.5.1 through 3.5.11. It is noted that these cross sections were prepared to represent stability conditions conservatively, and do not necessarily agree with cross sections at the same locations prepared for seepage analyses.

The calculated factors of safety for the existing condition toward the island and toward the slough were 1.40 and 1.34, respectively (Figure 3.5.1 and 3.5.2). The calculated factor of safety for the end-of-construction condition toward the island, assuming that all fill were placed in one stage (which we do not advocate), was below 1.0 by both methods (Figures 3.5.3 and 3.5.4). This result indicates, as expected, that the placement of levee fill will have to be done in multiple stages (see next paragraph below). We did not analyze the stability of the slope facing the slough for this condition because it would be similar to the existing condition described previously. The calculated factor of safety for the long-term post-construction condition toward the island was 1.82 (Figure 3.5.5). It is noted that, for potential failure surface through the lower portion of the island side slope, a factor of safety of 1.56 was calculated for this long-term condition. For the long-term post construction condition we performed an additional analysis using a lower water table in the peat layer under the levee. The reason for selecting this case was to model a situation where the free-field peat layer is so pervious that water runs through it and into the lower sand layer rather than runs across the peat layer under the levee. The factor of safety increased by about 45% over the higher water table case. The calculated factor of safety for the long-term post-construction stability toward the slough was 1.12 (Figure 3.5.6). The calculated factor of safety for the sudden drawdown condition toward the island was 1.18 (Figure 3.5.9). The results of pseudo-static stability analyses were K_y values of 0.027g toward the slough (Figure 3.5.8) and 0.151g toward the island (Figure 3.5.7).

To review end-of-construction stability further, we also calculated a factor of safety for a first-stage fill to elevation –2 feet. This case had an end-of-construction factor of safety just below 1.0 (Figure 3.5.10), demonstrating that a somewhat lower first stage fill could be designed to have adequate stability.

Table 3.5.2 summarizes the results of the slope stability analyses for station 630+00 on Webb Tract using the second configuration of the new fill; i.e., 3:1 initial slope flattening to 10:1 slope at elevation –3 feet. Cross sections showing various conditions along with the potential failure surfaces toward the island and the slough are shown in Figures 3.5.12 through 3.5.18. The calculated factor of safety for the end-of-construction condition toward the island was 1.12. This result indicates that this fill configuration is more stable than the first fill configuration in regard to end-of-construction stability. The factors of safety calculated for the other conditions are comparable to those calculated for the first configuration.

Table 3.5.3 summarizes the results of the slope stability analyses for station 160+00 on the Webb Tract using the first configuration of the new fill; i.e., uniform 5:1 slope. Cross sections showing various conditions along with the potential failure surfaces toward the island and the slough are shown in Figures 3.5.19 through 3.5.27. The calculated factors of safety for the existing condition toward the island and toward the slough were 1.24 and 1.29, respectively. The calculated factor of safety for the end-of-construction condition toward the island was below 1.0. This result is consistent with the previous results. The calculated factor of safety for the long-term post-construction condition toward the island was 1.57. The calculated factor of safety for the sudden drawdown condition toward the island was 0.88. Calculated K_y values were similar to those for the section at Station 630+00.

3.5.2 Bacon Island

The soil parameters used in the analysis are presented in Section 3.3.4 of this report. Table 3.5.4 summarizes the results of the slope stability analyses for station 25+00 using the first configuration of the new fill; i.e., uniform 5:1 slope. Cross sections showing various conditions along with the potential failure surfaces toward the island and the slough are shown in Figures 3.5.28 through 3.5.36. The calculated factors of safety for the existing condition toward the island and slough were 1.23 and 1.48, respectively. The calculated factor of safety for the end-of-construction condition toward the island was below 1.0. This result is consistent with the previous results. The calculated factor of safety for the long-term post-construction condition toward the island was 1.63. It is noted that, for a potential failure surface through the lower portion of the island side slope, a factor of safety of 1.40 was calculated for this long-term condition. The calculated factor of safety for the sudden drawdown condition toward the island was 1.07.

Table 3.5.5 summarizes the results of the slope stability analyses for station 265+00 using the first configuration of the new fill; i.e., uniform 5:1 slope. Cross sections showing various conditions along with the potential failure surfaces toward the island and the slough are shown in Figures 3.5.37 through 3.5.45. The calculated factors of safety for this section were comparable to those calculated for the section at Sta. 25+00.

For both stations 25+00 and 265+00, K_y values were computed for two different crest widths as shown in Tables 3.5.4. and 3.5.5. As expected, higher K_y values were computed for the wider crest. The K_y values of the slope facing the slough were identical for the two crest widths.

3.5.3 Summary of Static Slope Stability Analyses Results

A summary of the calculated factors of safety for the four representative cross sections and various conditions is presented in Table 3.5.6.

The factors of safety calculated for the existing condition toward the reservoir island range between 1.2 and 1.4. The lowest and highest factors of safety were calculated for Bacon Island Sta. 265+00 and Webb Tract Sta. 630+00, respectively. The results of the analyses indicate that the factor of safety decreases with increasing thickness of the peat layer. The factors of safety calculated for the existing condition toward the slough ranged between 1.3 and 1.5.

End-of-construction stability was evaluated to check whether the levee strengthening could be constructed in a single stage. As expected, most of the calculated factors of safety were below 1.0. These results indicate, as expected, that placement of the levee-strengthening fill will have to be done in several stages to prevent slope failures. One calculation made for a first-stage fill for a multi-slope construction showed a higher factor of safety than that for single-stage construction. The fill construction stages and their scheduling will have to be carefully selected during final design to prevent stability failures during construction.

The analyses of the long-term post construction conditions indicate that on the reservoir side the four representative sections had calculated factors of safety in the range of 1.6 to 1.8. The factor of safety increased by about 30% by widening the crest and flattening the slope to 5:1 when compared to the existing slope configuration. For the second fill configuration; i.e., 3:1 initial slopes flattening to 10:1 slope at elevation –3 feet, the factor of safety increased by about 20% over the existing slope configuration. It is noted that lower island-side factors of safety were calculated for potential failure surfaces through the lower portion of the interior slope of the sections. However, this type of failure does not significantly affect the integrity of the levee system. Therefore, we did not include these factors of safety in Table 3.5.6.

The long-term with-project condition toward the slough for the four representative sections had calculated factors of safety in the range of 1.1 to 1.3. The factor of safety with project was about 15% lower than for the present condition. A similar result was calculated for the second fill configuration. This reduction in factor of safety is due primarily to raising the reservoir water level up to elevation +6 feet, which creates seepage forces toward the slough.

The analyses of the sudden drawdown condition for the four representative sections toward the reservoir island indicated calculated factors of safety in the range of 0.9 to 1.2. It is noted that based on the proposed reservoir operation an instantaneous water drawdown is not feasible except in an emergency drawdown. Hence, the calculated factors of safety for the sudden drawdown condition are conservative.

We calculated the yield acceleration, K_y, which corresponds to a pseudo-static force producing a factor of safety of unity, for the representative sections. K_y values for the slopes facing the slough and the interior island were calculated. The water level conditions used in this analysis are presented in Section 3.3.3 of this report. The K_y values obtained are summarized in Table 3.5.6. The calculated K_y values toward the slough were between 0.017 and 0.048, while K_y values toward the island were between 0.114 and 0.151. The lower K_y values toward the slough are consistent with the lower factors of safety calculated for the slough side. This is generally due to the fact that the slopes on the slough side are steeper and irregular due to erosion effects; also, the reservoir water creates seepage forces toward the slough.

3.5.4 Static Stability Criteria and their Application to DW Project

Numerical criteria for the calculated factors of safety for static stability from several sources are summarized in Table 3.5.7. Geometric criteria from several other sources are presented in Figure 3.5.46.

It is not obvious which criterion or set of criteria should be used to judge the adequacy of the stability of the Delta Wetlands reservoir islands at this stage. It could be judged on the basis of (in order of increasing conservatism):

- the PL84-99 geometric criteria, or
- the Corps of Engineers' Delta-specific criteria,

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- the Corps' non-Delta-specific criteria, or
- Criteria of Association of State Dam Safety Officials (ASDSO).

The selection of applicable criteria could be based on the significance of the project; the consequences of failure (economic, environmental and other); the jurisdictional status of the reservoir under California Division of Safety of Dams (DSOD); and possibly other factors.

This is a significant project, in terms of the land area it encompasses, the investments in it, and the environmental and water-supply implications.

The consequences of failure depend on the type of failure that might occur. The most likely types (though all are unlikely) include:

- Failure of the reservoir island levee into the channel, slough or river, with full reservoir
- Failure of the reservoir island levee into the reservoir island, with the reservoir low or empty
- Failure of the adjacent island's levee due to seepage effects attributable to the reservoir island.

The first type of failure is judged least consequential. The loss is largely limited to the DW project in the form of loss of sellable water and required repair of the damages. In addition, development of a levee breach and sudden outflow of the stored waters into the channel may impact the adjacent island's levee and structures floating in the channel. This eventuality is discussed in Section 3.11.

Potential causes of an outward failure of the reservoir levee with full reservoir are long-term stability failure toward the slough, and potential earthquake effects with full reservoir. We judge that the factors of safety for the long-term failure toward the slough are marginal, and that the potential earthquake displacements in this direction are larger than what is generally considered as acceptable (see Section 3.6). One method to improve these situations is to flatten or otherwise strengthen the slough-side slope. However, this would require disturbing that slope, which may be difficult to have permitted because of environmental issues. Another potential method is to construct a rock toe buttress in the slough. A third method, that we recommended, is to provide a wide levee crest, such that slumping off of a section, say of 10 feet, would still leave enough levee crest width to provide a capable levee until repairs could be made.

The other two potential levee failures would have serious environmental consequences due to the large inflow of brackish water into the Delta, beside significant economic losses due to large repair costs and loss of beneficial use. These events clearly should be protected against with a significant margin.

The second type of failure, failure of the reservoir levee with reservoir empty, is considered adequately protected against, with high factors of safety for long-term failure into the reservoir island and adequate factors of safety for sudden drawdown at most locations. At some locations the levee geometry may need to be adjusted to provide an adequate factor of safety against sudden drawdown. Further, a large-scale stability failure during levee strengthening construction must be avoided by carefully-controlled staged construction.

Failure of the adjacent island's levee would be due to seepage effects and must be protected against by rigorous monitoring combined with application of the significance criteria. This is discussed in Sections 2.4 and 2.5. The monitoring methods and application of significance criteria should be periodically reevaluated and adjusted as may be indicated.

In addition to these long-term failures, it is important that end-of-construction failures be avoided. Such failures, although they would be unlikely to lead to levee breaches, would require significant extra effort to repair and would have the potential to delay construction.

3.6 SEISMIC-INDUCED LEVEE DEFORMATIONS AND GEOLOGIC HAZARDS

3.6.1 Objective

This section summarizes the analysis results of the seismic-induced levee deformations and geologic hazards for the proposed reservoir islands of the Delta Wetlands project. The analysis was performed for the proposed levee final design.

The study included the evaluation of the levees' seismic responses and deformations and the assessment of geologic hazard under the design earthquake ground motions. The details of the analysis approaches and results are presented in Appendix A.

3.6.2 Design Earthquake Ground Motions

Two horizontal earthquake acceleration time histories recorded during past earthquakes were selected for the analysis. These records were from the 1992 Landers and the 1987 Whittier Narrows earthquakes. The following table lists these selected motions along with the names of the recording stations, their closest distances from the rupture planes and recorded peak accelerations.

Summary of the Earthquake Records Used in the Dynamic Response Analysis

Earthquake	M_w	Recording Station			Comp	Recorded PGA (g)
		Distance (km)	Station #	Site Condition		
1987 Whittier Narrows	6.0	18	24402[b]	Soil[a]	90°	0.15
1992 Landers	7.3	64	24577[c]	Soil[a]	0°	0.11

Note : a = Deep Stiff Soil Site
b = Altadena – Eaton Canyon Station
c = Fort Irwin Station

The selected acceleration time histories were then modified to match the design earthquake response spectrum. Results from the recent CALFED study on the seismic hazard and levee failure probability of the Delta project were used to construct the design response spectrum (CALFED, 1999). A hazard exposure level corresponding to a 10% probability of exceedance in 50 years was selected for the design ground motions. This hazard exposure level results in a return period of about 475 years (or annual frequency of occurrence of 2.1×10^{-3}) and is consistent with the requirement adopted by the 1997 Uniform Building Code (UBC).

For the assessment of geologic hazards, two earthquake design criteria were used: earthquakes with magnitude (M_w) 6 and peak ground acceleration of 0.25g, and magnitude (M_w) 7.7 and peak ground acceleration of 0.13g. These ground motions represent the local and distant controlling seismic events and are consistent with the results of the CALFED study (CALFED, 1999).

3.6.3 Earthquake-Induced Levee Deformations

3.6.3.1 Dynamic Response Analysis

Four levee cross sections were analyzed for the two proposed reservoir islands: two cross sections for Webb Tract and two cross sections for Bacon Island. These selected cross sections are those used in the static and pseudo-static slope stability analyses (see Section 3.3.2). The results of dynamic response analysis are presented in terms of the average horizontal accelerations, which represent the seismic-induced inertia accelerations acting on the sliding masses and were used in the deformation analysis.

The computed average horizontal accelerations (K_{ave}) for the critical slide masses are shown in Figures A.6.11 and A.6.12, Figures A.6.16 and A.6.17, Figures A.6.21 and A.6.22, and Figures A.6.26 and A.6.27 of Appendix A. The peak values of these average horizontal accelerations (K_{max}) are tabulated in the following table.

Calculated Seismic-induced Slope Deformations

Critical Slide	K_y (g)	K_{max} (g)	Max Deformation (ft) Newmark[1]	Max Deformation (ft) Makdisi & Seed[2]
Webb Tract at St. 160+00				
Reservoir-side Slope Crest Slide	0.114	0.40	2.0	0.5-1.5
Slough-side Slope Crest Slide	0.025	0.21	3.5	0.5-3.5
Webb Tract at St. 630+00				
Reservoir-side Slope Crest Slide	0.151	0.36	1.5	0-1.0
Slough-side Slope Crest Slide	0.027	0.26	4.0	1.0-4.0
Bacon Island at St. 25+00				
Reservoir-side Slope Upper Toe Slide	0.148	0.47	3.5	0.5-1.0
Slough-side Slope Crest Slide	0.048	0.31	3.5	0.5-3.0

Critical Slide	K_y	K_{max}	Max Deformation (ft)	
	(g)	(g)	Newmark[1]	Makdisi & Seed[2]
Bacon Island at St. 265+00				
Reservoir-side Slope				
Crest Slide	0.133	0.36	1.5	0.5-1.0
Slough-side Slope				
Crest Slide	0.0385	0.28	3.5	0.5-3.0

Note: 1: Newmark Double Integration Method (1965)
2: Makdisi and Seed Simplified Method (1978)

3.6.3.2 Levee Deformations

The calculated deformations of the selected critical slide masses of the levees on the Webb Tract and Bacon Island are tabulated in Table 3.6.2. These deformations were estimated using both the Newmark Double Integration Method (Newmark, 1965) and the Simplified Procedure of Makdisi and Seed (Makdisi and Seed, 1978) for comparison. In estimating the deformation, we rounded the calculated deformation to the nearest ½ foot.

The results of analysis indicate that the slough-side slopes may experience up to about 4 feet of deformations. Smaller deformations can be expected for the reservoir-side slopes, due to the increased stability provided by the proposed new fills, and flatter slopes.

3.6.4 Earthquake-Induced Geologic Hazards

The seismic-induced geologic hazards assessed for this study included liquefaction, loss of bearing capacity, settlement and levee overtopping.

3.6.4.1 Liquefaction

We used the data from HLA's exploratory borings (HLA, 1987) to assess the potential for liquefaction during the occurrence of the design ground motions.

The evaluation procedure for liquefaction susceptibility proposed by the National Center for Earthquake Engineering Research (NCEER) Workshop (Youd and Idriss,1997) was utilized for this study. We applied the corrections to the measured penetration blow counts, as recommended by the NCEER procedure.

The results of the analyses indicate that a few pockets of potentially liquefiable soil deposit may exist in the levees and foundation soils. We believe, however, that these liquefiable soil pockets are confined in limited areas and therefore are expected to have negligible adverse effects on the stability of the levees.

3.6.4.2 Loss of Bearing Capacity

Seismic-induced bearing capacity loss/reduction is associated mainly with the occurrence of liquefaction or pore water pressure generation. The reduction may be substantial for shallow foundations supported on or near the liquefied soils. Based on the results of the liquefaction

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evaluation and the absence of shallow foundations at the project site, we judge that the risk of loss of bearing capacity that may affect levee performance is insignificant.

3.6.4.3 Dynamic Soil Compaction

Similar to the seismic-induced bearing capacity loss, the dynamic soil compaction would only be significant following the occurrence of extensive liquefaction. Since the potential for liquefaction at the project islands is limited to a few isolated pockets, we judge that the potential for dynamic soil compaction (settlement) at these islands is negligible.

3.6.4.4 Levee Overtopping During Seismic-Induced Seiche

Earthquakes can cause overtopping of levee due to three primary mechanisms: Landslide generated waves, static displacement of the reservoir or dynamic displacement of the reservoir. Both the landslide induced waves and static displacement of the reservoir are not expected to occur at the project reservoirs.

Records for past occurrences of seiche are generally incomplete. The largest seiche reported in the United States was in Lake Ouachita in Arkansas with a maximum amplitude of about 0.44 m (1.5 feet). We have estimated the amplitudes of seismic-induced waves (dynamic displacement of reservoir water) using the procedure of United States Committee on Large Dams (USCOLD, 1995). The results of the analysis indicate a negligible amplitude of seismic-induced wave (less than 1 foot). It should be noted that this procedure was developed for a limited body of water (tanks, dams) and has been assumed to be applicable to the DW Project reservoirs.

3.7 EXPECTED SETTLEMENTS AND EFFECTS ON STABILITY

A settlement analysis was performed for the section at Sta. 630+00 in Webb Tract to study the effect of consolidation settlement on stability and to estimate the required fill volume. Two types of settlements resulting from the levee strengthening were estimated. The first consists of the long-term consolidation settlement that corresponds to the slow volume change associated with the dissipation of excess pore pressures as the soil is subjected to a sustained load. The second consists of the secondary consolidation settlement that corresponds to the long-term creep of peat.

The consolidation settlement was estimated using analysis based on laboratory one-dimensional consolidation tests. The tests were performed by HLA on peat samples acquired from Delta Wetlands Islands, presented in HLA (1989), Appendices A & B, Vol. 2. The evaluation of the total consolidation settlement was based on relevant parameters including preconsolidation pressure, stress increase due to added fill, in-situ stress conditions, and compression indices for the virgin loading curve and the unloading-reloading curve. The coefficient of consolidation was obtained from the laboratory time rate of consolidation tests for various levels of load increments. A summary of the consolidation parameters used in the analysis is presented in the table below:

SUMMARY OF CONSOLIDATION PARAMETERS FOR PEAT

Material	C_c	C_r	e_o	C_α	c_v (ft^2/yr)	σ_p' (psf)
Peat	3.8	0.69	8.428	0.17	75	300

where

- C_c is the compression index for the virgin loading curve, calculated from a one-dimensional consolidation test on a peat sample from 30-foot depth at Webb Tract (see Plate 13 of HLA 1989) and validated by comparison to similar data.
- C_r is the compression index for the reloading curve.
- C_α is the secondary compression index, calculated using a C_α / C_c ratio of 0.045.
- e_o is the initial void ratio, calculated from the initial water content of the peat.
- c_v is the coefficient of consolidation corresponding to a vertical effective stress of about 1000 psf (Plate 13, HLA [1992e]).
- σ_p' is the average preconsolidation pressure of peat, estimated at 300 psf based on laboratory tests.

We assumed that fill was placed instantaneously and that strength gain occurs as peat consolidates and pore pressure dissipates due to the load imposed by the fill. The consolidation settlement in the peat under the fill load will require the addition of fill to maintain the required fill height as settlement occurs. The settlement analysis for the fill construction was made iteratively until the final levee height was reached eventually. A table indicating the maximum consolidation settlements for each iterative step is presented below. Figure 3.7.1 shows the estimated settlement profile after the first stage of load application. We assumed settlement due to secondary compression would take place only after final construction is completed.

EXPECTED MAXIMUM SETTLEMENT

Iteration #	Expected Maximum Settlement (feet)
1	5.10
2	2.25
3	1.00
Expected Total	9.0

While the settlement calculation was based on instantaneous loading, the actual construction will be performed in stages to prevent adverse stability conditions. We anticipate that three to four stages of construction will be required to place the additional fill material. Each construction stage will take about one to two years to achieve required consolidation settlement and gain in strength to allow the next stage to be constructed. Therefore, the projected construction time to raise and widen the levee may be 4 to 6 years.

We then calculated the factors of safety for the long-term condition using the deformed geometry. The cross sections showing water conditions on both slough and island sides along with the potential failure surfaces toward the island and the slough are shown in Figures 3.7.2 and 3.8.3. The calculated factor of safety increased by about 6% for the slope toward the island when using the deformed levee cross section. There was no change in the calculated factor of safety for the slope toward the slough which remained at 1.12.

3.8 WAVE HEIGHT ESTIMATES AND EROSION ASSESSMENT OF LEVEES

Wave runup analyses for Bacon Island and Webb Tract were made to evaluate freeboard and erosion potential characteristics for the levees when the islands are used as storage reservoirs. Wave runup is defined as the vertical height above stillwater level to which water from an incident wave will run up the face of a structure. The analyses involved estimating wave runup characteristics from wind velocities, reservoir fetch, and levee geometry.

Wind velocities for the "fastest mile of record" were obtained from generalized charts published by the U.S. Army Corps of Engineers (USACE, 1976) and U.S. Bureau of Reclamation (USBR, 1981). These data indicate that the estimated fastest mile of record wind velocities over land at elevation 25 feet for winter, spring, summer, and fall are 58, 52, 40, and 60 miles per hour, respectively. The effective fetch, F, of the islands were calculated using procedures in USACE's "Shore Protection Manual" (1984). The largest effective fetch for Bacon Island and Webb Tract are 3.15 miles and 2.83 miles, respectively. Analyses were made assuming levee bank slopes of 3H:1V and 5H:1V on the reservoir side of the levees. Rip-rap on the face of the levee was considered in the calculations.

An estimate of the reservoir setup resulting from winds was also made. Reservoir setup is a general tilting of the reservoir surface due to shear stresses caused by winds.

Using the procedures presented above, wave runup for Bacon Island and Webb Tract for the most severe wind conditions (Fall) were calculated to be 6.4 feet and 6.1 feet, respectively, for the 3H:1V levee bank slope and 4.0 feet and 3.8 feet, respectively, for the 5H:1V levee bank slope. Wave runup of these magnitudes will pose a significant potential for erosion of the levees absent erosion protection. If rip-rap is used on the bank slopes, runup would be reduced to about 55% of these values and would also greatly reduce the potential for erosion.

In addition to runup, wind shear will also cause a setup in the reservoir. Setup for Bacon Island and Webb Tract was calculated to be 0.38 feet and 0.34 feet, respectively.

Results of these analyses were compared to general wave runup estimates published in the California Department of Water Resources' Bulletin 192-82 (DWR, 1982) and are consistent with them.

3.9 BORROW REQUIREMENTS

We estimate that as much as 4 million cubic yards of fill may be needed at Webb Tract, and slightly more than that at Bacon Island, to bring and maintain the islands' levees to the strengthened cross section. These estimates include not only the initial fill quantity but also the additional quantities required later to restore and continue restoring the levees to the specified configuration to compensate for long-term settlement.

DW plans to use on-island borrow material. The seepage considerations described in Section 2.3 have indicated that borrow pits can be excavated down to and into the underlying sand layer without any discernible effect on seepage conditions that might affect neighboring islands if the borrow area is at least 800 to 1000 feet set back from the levee.

We understand that there are numerous sand mounds on Webb Tract, which could be used as borrow, if necessary with some fines blended in. Assuming that five percent of the island's area of about 5500 acres has surficial sand and is located more than 1000 feet from the levee, a borrow depth of about 9 feet would be sufficient to provide the needed borrow volume. It is obvious that enough borrow is available, either in the sand mounds or below the peat layer, to borrow the required fill quantity.

We are not aware of sand mounds on Bacon Island. Approximately 3600 acres of the island's total land area of about 5500 acres is located more than 1000 feet from the levee. Using one tenth of this area for borrow would require a borrow depth into the sand of about 7.5 feet to generate the required fill quantity. This type of borrowing could be done with relatively simple dewatering.

It is concluded that it will be easy to mobilize the required amounts of borrow fill needed to upgrade the levees on each island, with nominal dewatering.

3.10 EFFECT OF INTERCEPTOR WELLS ON SLOPE STABILITY

The results of the evaluation of the interceptors wells presented in Section 2.3 indicated that 6-inch diameter wells spaced at about 160 feet or larger were generally adequate to mitigate the potential underseepage and flooding at the neighboring islands. From a stability viewpoint the wells' size and spacing is such that the ratio of the area occupied by the well over the tributary area of the levee is very small or insignificant. Therefore, the presence of the wells would not have any significant impact on the stability of the levees and supporting foundation.

However, the high rate of continuous pumping during reservoir operation should be considered carefully in relation to potential internal erosion/piping. An inadequately designed and constructed filter system may cause internal erosion and piping which may create cavities under the levees and possibly result in the formation of sink holes and deterioration of the levee foundation. The design, construction, and quality control during installation and development of the interceptor wells should be addressed carefully in the design and implementation of the wells system. Of particular interest are the reliability and compatibility of the filter packing between the base soil (aquifer gradation) and the well screen's schedule. This may require a careful site-specific characterization of the aquifer properties (grain size distribution at various locations and various depths). Standard procedures with detailed guidelines for design and construction of pumping wells are widely available and used in the industry. The documentation of the wells' design details should be provided in the design phase for the DW project.

One effect of internal soil erosion around extraction wells is a gradual silting up of the wells, which reduces their hydraulic effectiveness. This effect can be controlled by re-developing the well. This may be done periodically or in response to evidence of lack of effectiveness of the well. For this reason it would be advisable to be able to measure flow rates in individual wells, such that lack of performance can be identified and corrected.

A second set of related potential effects of internal soil erosion around extraction wells may occur, if the internal erosion process is ongoing for an extended time. This can lead to potential settlements in the vicinity of the well and potential development of a meta-stable (half-stable) soil structure, which could suddenly collapse, with or without provocation, and cause significant levee settlement and potential levee instability. This is a major reason why silting up of wells cannot be tolerated on this project. Measures against this occurring, after well construction, are monitoring of individual wells' flows to judge well pumping efficiency, and tracking of redevelopment of wells; if it were to occur at frequent intervals, it would be a sign of loss of fines. In severe cases the well may have to be abandoned and rebuilt using appropriate construction methods and materials to minimize soil loss in the future.

3.11 LEVEE BREACH ANALYSIS AND PROJECT ABANDONMENT

In this section the potential consequences of a sudden levee breach and project abandonment on adjacent Delta islands are discussed. It is noted that a levee breach has a very low likelihood of occurring, provided seepage and stability issues are addressed as discussed in this report.

The following is a summary of the hydraulic analysis that was conducted to determine maximum bank velocities along the downstream levee opposite the breach opening between Webb Tract and Bradford Island in Delta Wetlands:

- The breach analysis location between Webb Tract and Bradford Island was selected because it is one of the shortest distances between a reservoir island and a neighboring island (it represents the most adverse impact from a levee break).
- The assumed water elevation in the adjacent slough was considered to be at –2 feet, while the reservoir level in the island was at elevation +6 feet.

To judge the potential effects of a failure of the reservoir island (filled to the maximum elevation of +6 feet) into the channel, the hydraulic effects of a potential levee breach were analyzed. For assumed breach widths of 40, 80, 200 and 400 feet, respectively, the maximum resulting flow velocities along the opposite bank of the channel were calculated as 2, 9, 12 and 16 feet per second, respectively. The pattern of the calculated flow velocities for the 400-foot breach is illustrated in Figure 3.11.1. A 400-foot breach width is the widest expected breach width for this situation, based on breaching characteristics of dam failures (MacDonald and Langridge-Monopolis, 1984).

The 400-foot breach would cause a water level runup on the opposite shore up to elevation + 5.2 feet. This elevation would not present an overtopping threat. Further, this breach would have a calculated discharge rate of water out of the reservoir island of 123,000 cubic feet per second. At this discharge rate, the water surface elevation in the larger of the two reservoir islands would drop at an approximate rate of 1.6 feet per hour. Consequently, the highest flow velocity in the channel would be sustained for about 30 to 40 minutes, and would gradually decrease as the water level in the reservoir island dropped and the discharge rate decreased.

The flow rate of 16 feet per second along the opposite bank sustained for that duration would be expected to cause erosion of unprotected levee slopes, but would likely not cause severe damage to a rip-rapped levee slope. Floating structures and boats moored in the channel near the location of the levee breach would probably experience damages.

Abandonment of the project by its sponsors would leave project facilities designed for the planned use. Following are some thoughts on potential consequences of such abandonment. First and foremost, we do not expect an immediate threat to adjacent islands. Longer-term levee maintenance must be continued. The project's facilities could probably be converted back to serve traditional island uses. There would clearly be considerable time and effort required to re-start agriculture on the island. It is believed that the expense of this effort would be less than the land value of the island. Similarly, should the project be abandoned with a full reservoir, the value of the stored water should pay for discharging it to the Delta channels (using the project's facilities). The most unfavorable case would be if the project were abandoned after failure of a levee. The conversion in this case would require repairing the levee and pumping out the island in addition to the cost of any other damage that the levee breach may have caused. At any rate, it appears that project abandonment should be followed by re-conversion to agricultural use, that it is likely that such conversion could be done at no cost to the public, and that after conversion the new operators would maintain the levees to minimize the potential for future levee failures. This topic of project abandonment will deserve more detailed evaluation, primarily economic, to assess all probable eventualities.

Table 3.5.1
Results of Slope Stability Analyses
for Webb Tract Sta. 630+00
Fill with 5:1 Slope

Condition	Factor of Safety: Toward Slough	Factor of Safety: Toward Island	Remarks
Existing	- 1.34	1.40 -	Drained analysis
End of Construction, One Stage[a]	- -	0.92 0.85	Two-stage analysis S_u profile calculated using HLA's design curve for peat
Long-Term Condition	- 1.12	1.56[c] 1.82[d] -	Drained analysis
Seismic Loading	- 1.00 K_y= 0.027	1.00 K_y= 0.151 -	Two-stage analysis
Sudden Drawdown	-	1.18	Three-stage analysis
Staged loading, 1st Stage to Elev. –2 ft End of Construction[b]	 - -	 0.96 1.02	 Two-stage analysis S_u profile calculated using HLA's design curve for peat

Notes:
[a] Assuming one-stage construction, which we do not advocate. The factors of safety for the multiple-stage construction have not been calculated because these criteria need to be established during final design as to stage fill thickness and time sequencing.
[b] Result for first stage of multi-stage construction
[c] For small toe circle
[d] For large circle

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Table 3.5.2
Results of Slope Stability Analyses
for Webb Tract Sta. 630+00
Fill with 3:1 Slope Flattening to 10:1 Slope at Elev. –3 Feet

Condition	Factor of Safety Toward slough	Factor of Safety Toward island	Remarks
Existing	- 1.34	1.40 -	Drained analysis (same as 5:1 slope)
End of Construction[a]	- -	1.32 1.12	Two-stage analysis S_u profile calculated using HLA's design curve for peat
Long-term	- 1.12	1.26[b] 1.71[c] -	Drained analysis
Seismic Loading	- 1.00 K_y= 0.017	1.00 K_y= 0.144 -	Two-stage analysis
Sudden Drawdown	-	1.04	Three-stage analysis

Notes:
[a] Assuming one-stage construction, which we do not advocate. The factors of safety for the multiple-stage construction have not been calculated because these criteria need to be established during final design as to stage fill thickness and time sequencing.
[b] For small toe circle
[c] For large circle

Table 3.5.3
Results of Slope Stability Analyses
for Webb Tract Sta. 160+00
Fill with 5:1 Slope

Condition	Factor of Safety: Toward Slough	Factor of Safety: Toward Island	Remarks
Existing	- 1.29	1.24 -	Drained analysis
End of Construction[a]	- -	0.58 0.65	Two-stage analysis S_u profile calculated using HLA's design curve for peat
Long-term	- 1.13	1.36[b] 1.57[c] -	Drained analysis
Seismic Loading	- 1.00 K_y= 0.025	1.00 K_y= 0.114 -	Two-stage analysis
Sudden Drawdown	-	0.88	Three-stage analysis

Notes:

[a] Assuming one-stage construction, which we do not advocate. The factors of safety for the multiple-stage construction have not been calculated because these criteria need to be established during final design as to stage fill thickness and time sequencing.

[b] For small toe circle

[c] For large circle

Table 3.5.4
Results of Slope Stability Analyses
for Bacon Island Sta. 25+00
Fill with 5:1 Slope

Condition	Factor of Safety: Toward Slough	Factor of Safety: Toward Island	Remarks
Existing	- 1.48	1.23 -	Drained analysis
End of Construction[a]	- -	0.88 0.91	Two-stage analysis S_u profile calculated using HLA's design curve for peat
Long-term	- 1.33	1.40^b 1.63^c -	Drained analysis
Seismic Loading	- 1.00 K_y= 0.048	1.00 K_y= 0.128 K_y= 0.148 -	Two-stage analysis Crest width = 35 feet Crest width = 45 feet
Sudden Drawdown	-	1.07	Three-stage analysis

Notes:
[a] Assuming one-stage construction, which we do not advocate. The factors of safety for the multiple-stage construction have not been calculated because these criteria need to be established during final design as to stage fill thickness and time sequencing.
[b] For small toe circle
[c] For large circle

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Table 3.5.5
Results of Slope Stability Analyses
for Bacon Island Sta. 265+00
Fill with 5:1 Slope

Condition	Factor of Safety		Remarks
	Toward Slough	Toward Island	
Existing	- 1.49	1.21 -	Drained analysis
End of Construction[a]	- -	0.90 0.81	Two-stage analysis S_u profile calculated using HLA's design curve for peat
Long-term	- 1.23	1.31[b] 1.64[c] -	Drained analysis
Seismic Loading	- 1.00 K_y= 0.038	1.00 K_y= 0.115 K_y= 0.133 -	Two-stage analysis Crest width = 35 feet Crest width = 45 feet
Sudden Drawdown	-	0.98	Three-stage analysis

Notes:

[a] Assuming one-stage construction, which we do not advocate. The factors of safety for the multiple-stage construction have not been calculated because these criteria need to be established during final design as to stage fill thickness and time sequencing.

[b] For small toe circle

[c] For large circle

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TABLE 3.5.6
SUMMARY RESULTS OF SLOPE STABILITY ANALYSES

FILL WITH 5:1 SLOPE

Island Profile	Existing Condition		End of Construction[b]	Long-term Condition		Sudden Drawdown	K_y	
	Toward Island	Toward Slough	Toward Island	Toward Island	Toward Slough	Toward Island	Toward Island	Toward Slough
Webb Tract Sta. 160+00	1.24	1.29	0.62	1.57	1.12	0.88	0.114	0.025
Webb Tract Sta. 630+00	1.40	1.34	0.89	1.82	1.12	1.18	0.151	0.027
Bacon Sta. 25+00	1.23	1.48	0.90	1.63	1.33	1.07	0.128	0.048
Bacon Sta. 265+00	1.21	1.49	0.86	1.64	1.23	0.98	0.115	0.038

FILL WITH 3:1 SLOPE FLATTENING TO 10:1 SLOPE AT ELEV. –3 FEET

Webb Tract Sta. 630+00	1.40	1.34	1.22	1.71	1.12	1.04	0.144	0.017

Notes:

[a] Assuming one-stage construction, which we do not advocate. The factors of safety for the multiple-stage construction have not been calculated because these criteria need to be established during final design as to stage fill thickness and time sequencing.

[b] average of the two methods (two-stage method and S_u/p' method)

Table 3.5.7
Stability Criteria for Levees and
Factors of Safety for Dam Safety Evaluations

Case	Design Condition	Factors of Safety for Dam Safety Evaluations[1]	Minimum Levee Factor of Safety by USACE[2]	DWR Criteria for Levee Rehabs[4]	PL84-99 for Non-Federal Levees[5]
1	Immediately Following Construction	-	1.3	-	-
2	Sudden Drawdown	1.25	1.0	-	-
4	Long-term, Steady-State at Flood Stage	1.5	1.4	1.3	1.25
5	Seismic Loading (Pseudo-Static Analysis)	1.2[3]	1.0	-	-

Notes:

1. From ASDSO (1989).
2. From USACE (1978).
3. Deformation criteria are also used to satisfy that no excessive deformations occur.
4. From California DWR (1989).
5. From USACE (1988)

Table 3.6.1
Dynamic Soil Parameters Used in the Response Analysis

Description	Moist Unit Weight pcf	K_{2max}	Shear Wave Velocity ft/sec	Modulus and Damping Curves
Levee Materials				
New fills: sand	120	80	-	Sand[3]
Fills: sand	105-110	25	-	Sand[3]
Fills: soft clay	70	-	250	Clay[1]
Fills: silty sand with fat clay	110	25	-	Sand[3]
Fills: clay with peat	80	-	300	Clay[1]
Fills: silty clay with sand	110		450	Clay[2]
Peat	70	-	250	Peat[4]
Foundation Materials				
Sand	120-125	80	-	Sand[3]
Clay	127	-	700	Clay[2]

Note : 1: Relationships of Vucetic and Dobry (1991) for PI = 100
2: Relationships of Vucetic and Dobry (1991) for PI = 50
3: Relationships of Seed and Idriss (1970) for mean
4: Relationships of Boulanger et al (1997)

Table 3.6.2
Summary of the Earthquake Records Used in the Dynamic Response Analysis

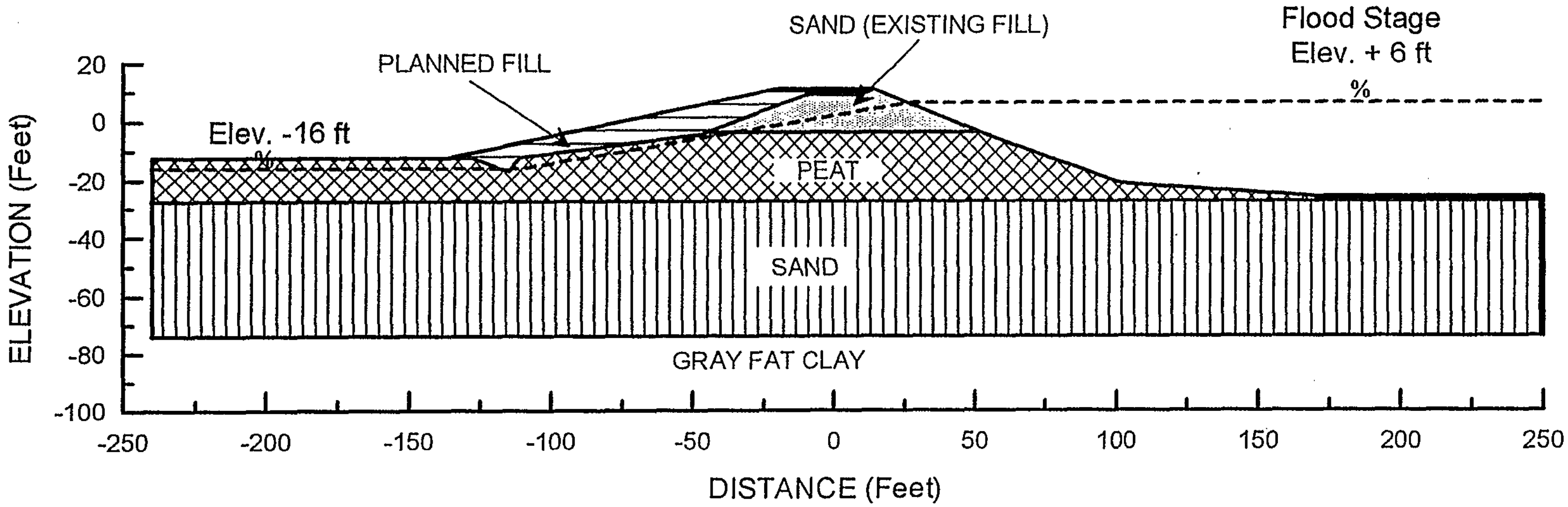
Earthquake	M_w	Recording Station: Distance (km)	Recording Station: Station #	Recording Station: Site Condition	Comp	Recorded PGA (g)
1987 Whittier Narrows	6.0	18	24402[b]	Soil[a]	90°	0.15
1992 Landers	7.3	64	24577[c]	Soil[a]	0°	0.11

Note : a = Deep Stiff Soil Site
b = Altadena – Eaton Canyon Station
c = Fort Irwin Station

Table 3.6.3
Calculated Seismic-induced Slope Deformations

Critical Slide	K_y (g)	K_{max} (g)	Max Deformation (ft)	
			Newmark[1]	Makdisi & Seed[2]
Webb Tract at St. 160+00				
Reservoir-side Slope				
Crest Slide[3]	0.114	0.40	2.0	0.5-1.5
Slough-side Slope				
Crest Slide[3]	0.025	0.21	3.5	0.5-3.5
Webb Tract at St. 630+00				
Reservoir-side Slope				
Crest Slide[4]	0.151	0.36	1.5	0-1.0
Slough-side Slope				
Crest Slide[4]	0.027	0.26	4.0	1.0-4.0
Bacon Island at St. 25+00				
Reservoir-side Slope				
Upper Toe Slide[5]	0.148	0.47	3.5	0.5-1.0
Slough-side Slope				
Crest Slide[5]	0.048	0.31	3.5	0.5-3.0
Bacon Island at St. 265+00				
Reservoir-side Slope				
Crest Slide[6]	0.133	0.36	1.5	0.5-1.0
Slough-side Slope				
Crest Slide[6]	0.0385	0.28	3.5	0.5-3.0

Note: 1: Newmark Double Integration Method (1965)
2: Makdisi and Seed Simplified Method (1978)
3: Refer to Figures 3.5.25 and 3.5.26.
4: Refer to Figures 3.5.7 and 3.5.8.
5: Refer to Figures 3.5.34 and 3.5.35.
6: Refer to Figures 3.5.43 and 3.5.44.



Soil Properties			
Material	γ_s (pcf)	c' (psf)	ϕ' (deg)
Clay with Peat and Sand	110	80	27
Peat	70	50	28
Sand	125	0	36
Planned Fill	120	0	34

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	**WEBB TRACT STA. 160+00 CROSS SECTION FOR STABILITY ANALYSIS**	**FIGURE 3.3.1**

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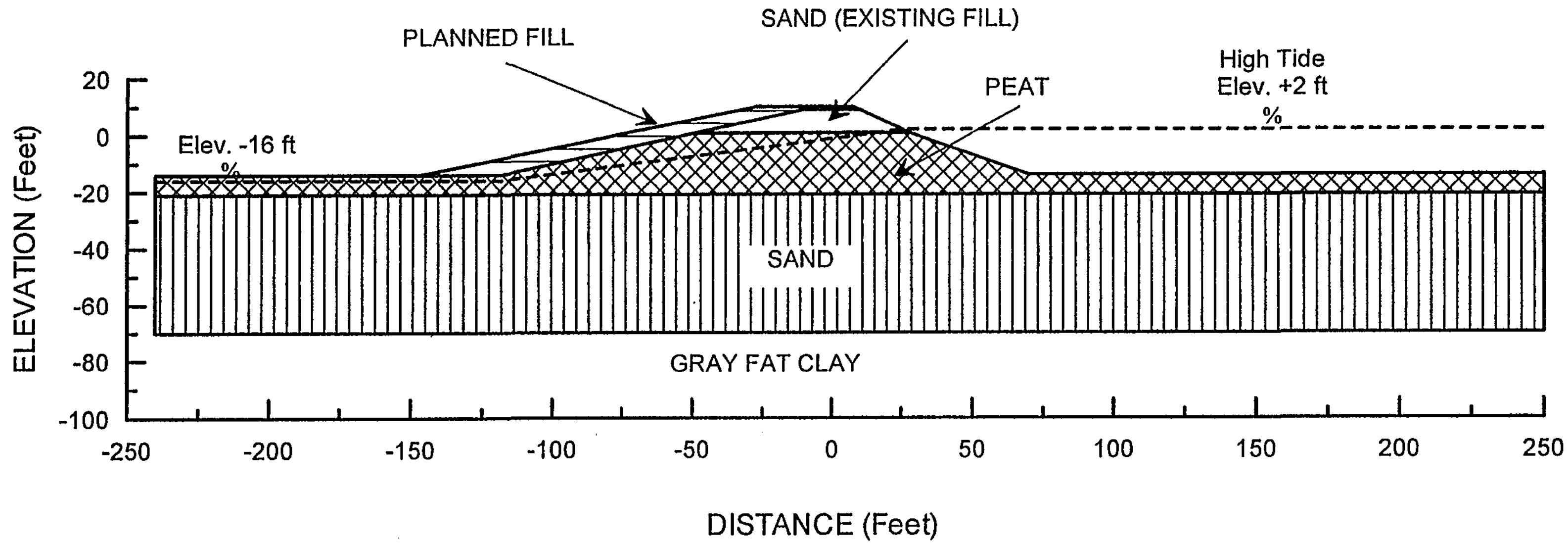

Soil Properties			
Material	γ_s (pcf)	c' (psf)	ϕ' (deg)
Sand Fill	110	0	32
Soft Clay Fill or Peat	70	50	28
Sand	125	0	36
Planned Fill	120	0	34

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	**WEBB TRACT STA. 630+00 CROSS SECTION FOR STABILITY ANALYSIS**	**FIGURE 3.3.2**

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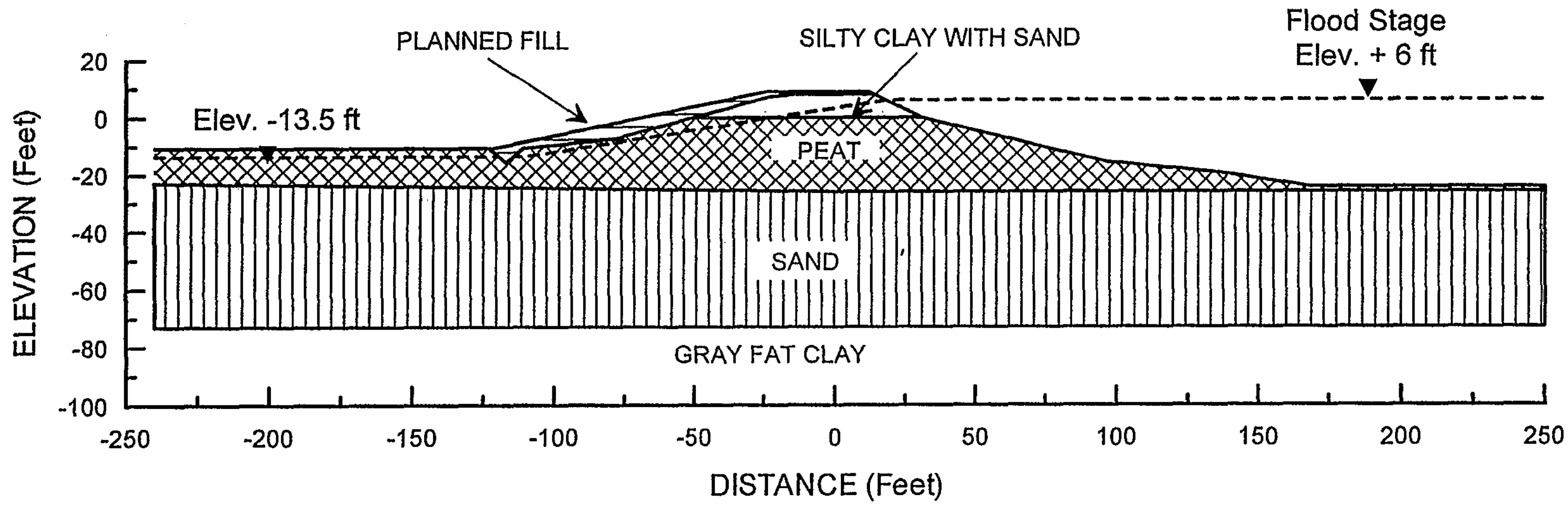
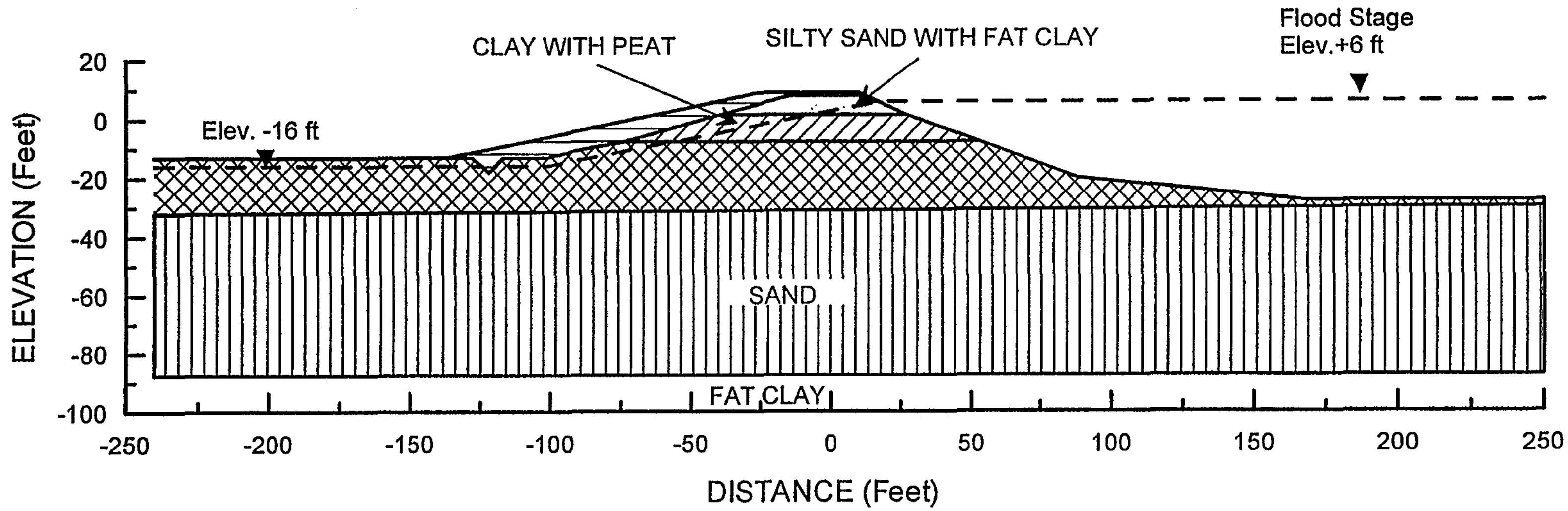


Soil Properties			
Material	γ_s (pcf)	c' (psf)	ϕ' (deg)
Silty Clay with Sand	110	80	27
Peat	70	50	28
Sand	125	0	36
Planned Fill	120	0	34

DELTA WETLANDS PROJECT	**BACON ISLAND STA. 25+00 CROSS SECTION FOR STABILITY ANALYSIS**	**FIGURE 3.3.3**
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/bacon25/bacon25.grf 12/3/09

C—063470



Soil Properties			
Material	γ_s (pcf)	c' (psf)	ϕ' (deg)
Silty sand with Fat Clay	110	0	32
Clay with Peat	80	100	28
Peat	70	50	28
Sand	120	0	37
Planned Fill	120	0	34

DELTA WETLANDS PROJECT	**BACON ISLAND STA. 265+00 CROSS SECTION FOR STABILITY ANALYSIS**	**FIGURE 3.3.4**
URS GREINER WOODWARD CLYDE		

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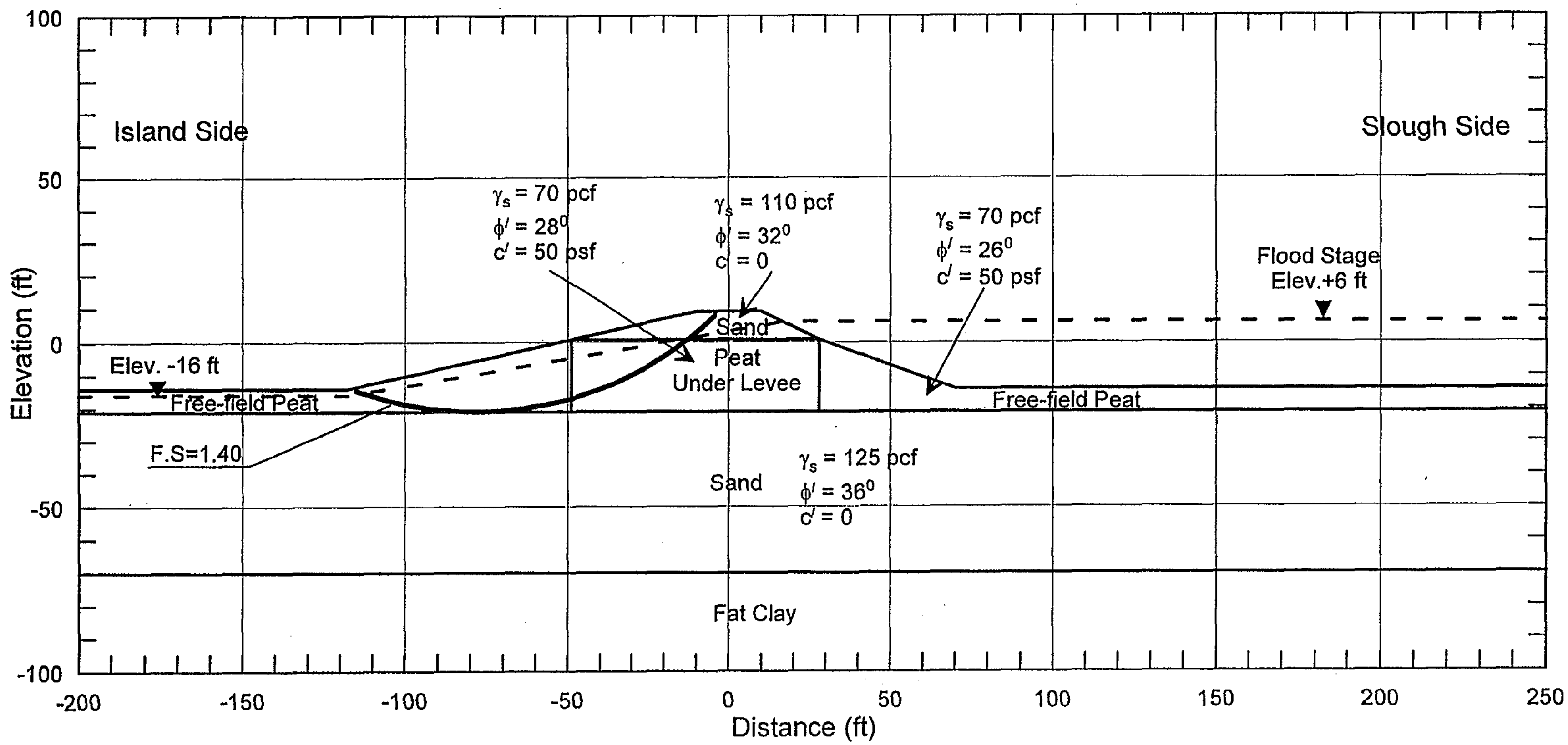

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS EXISTING CONDITION-INTO ISLAND	FIGURE 3.5.1
URS GREINER WOODWARD CLYDE		

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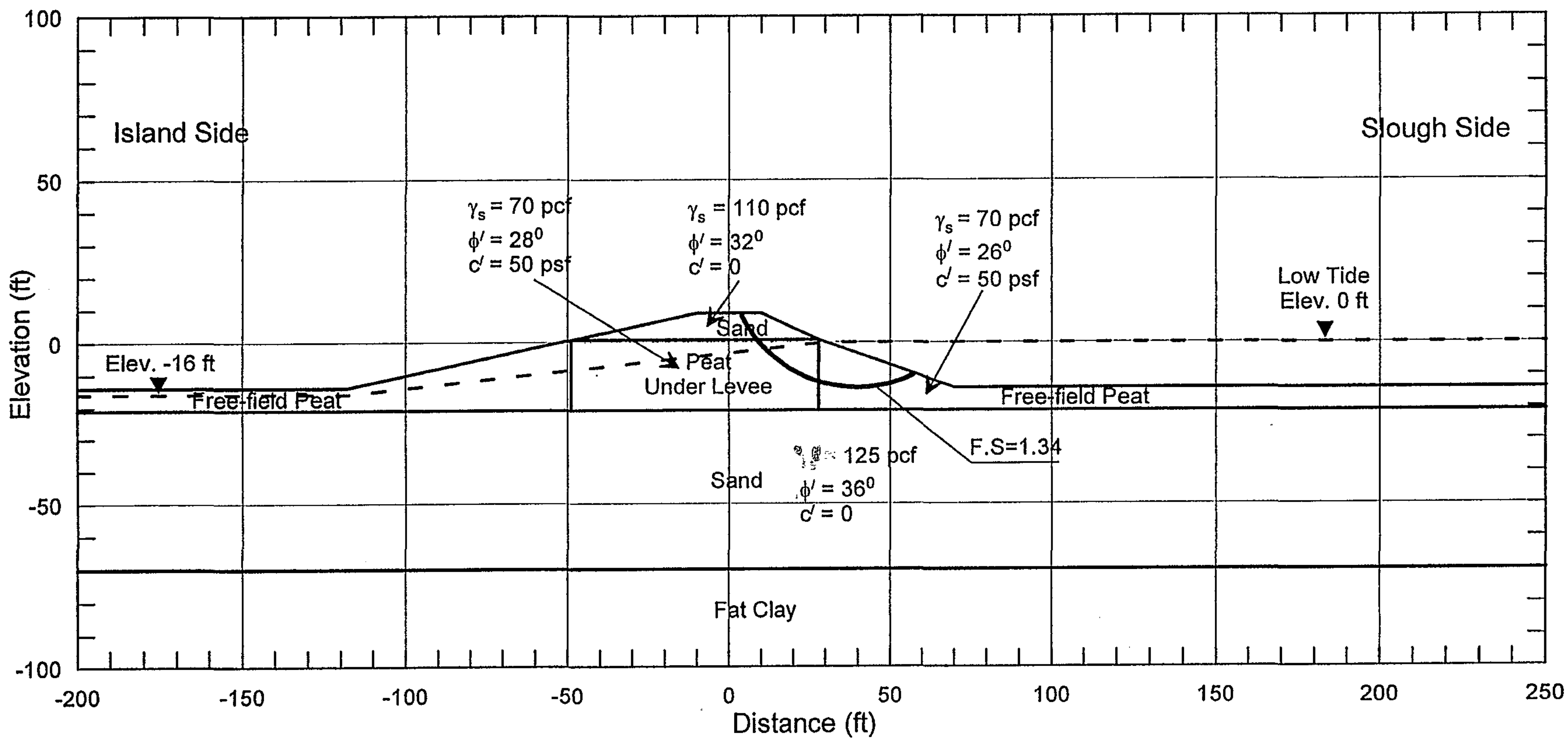

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	WEBB TRACT STA. 630+00 STABILITY ANALYSIS EXISTING CONDITION- TOWARD SLOUGH	FIGURE 3.5.2

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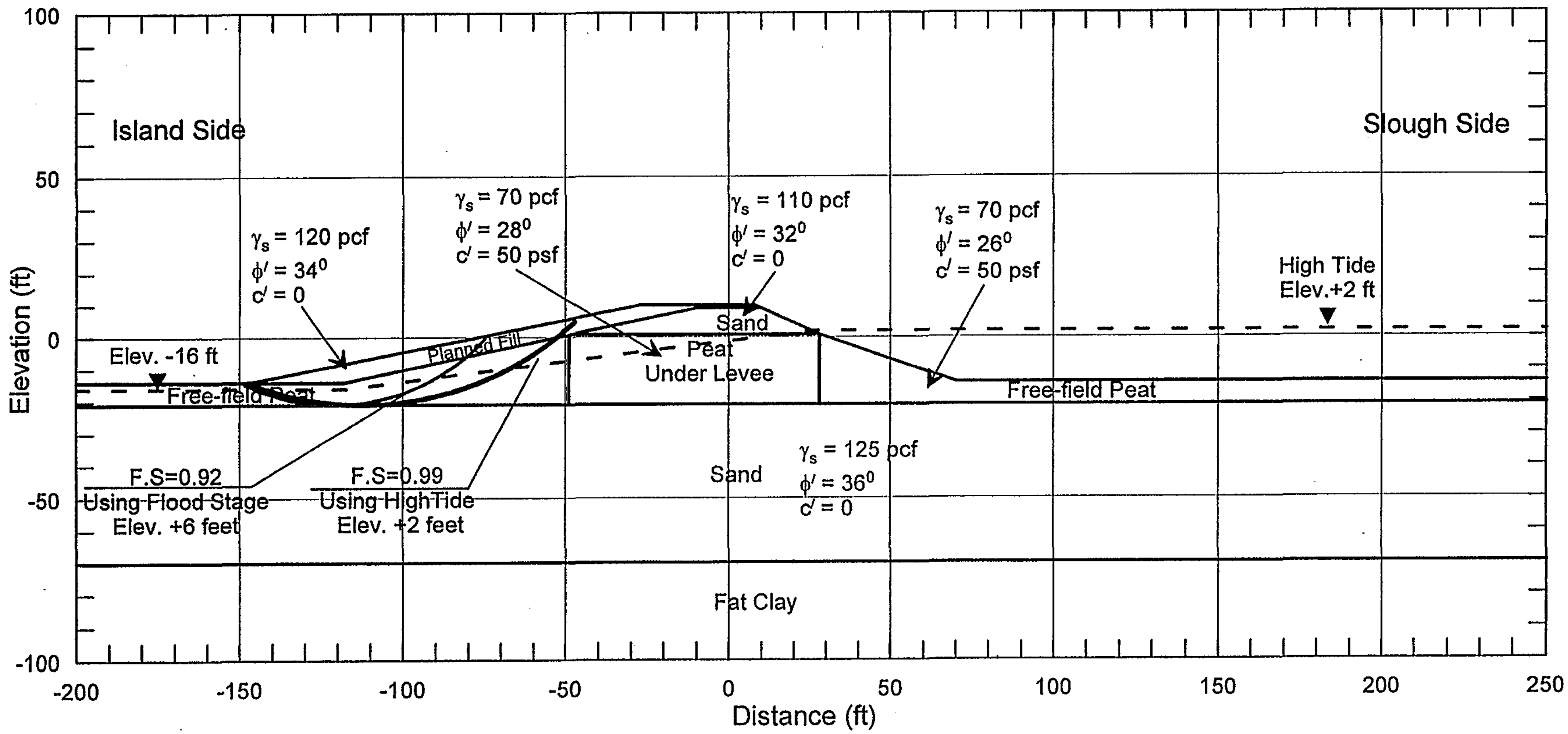
C—063473



DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS, TWO-STAGE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.3
URS GREINER WOODWARD CLYDE		

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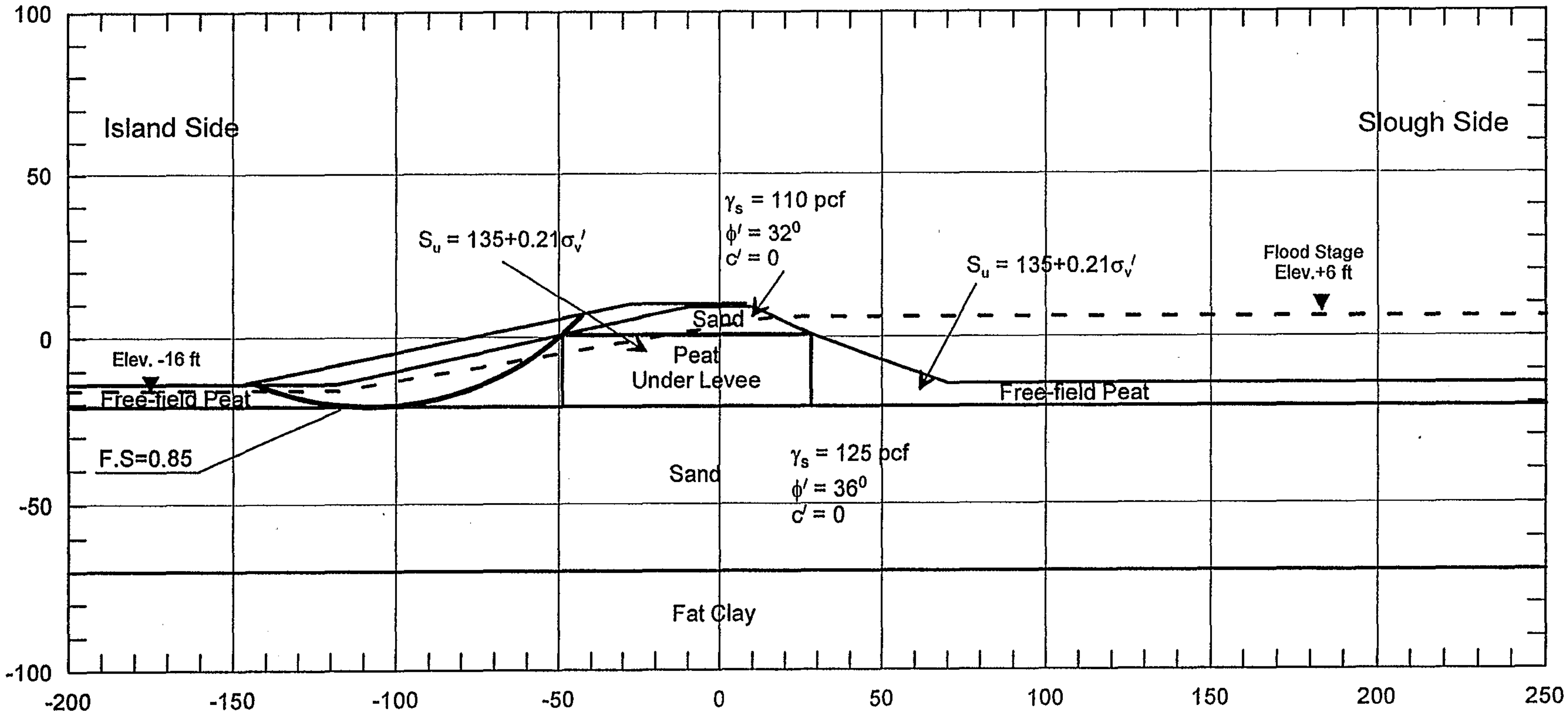
C-063474



DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS, S_u PROFILE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.4
URS GREINER WOODWARD CLYDE		

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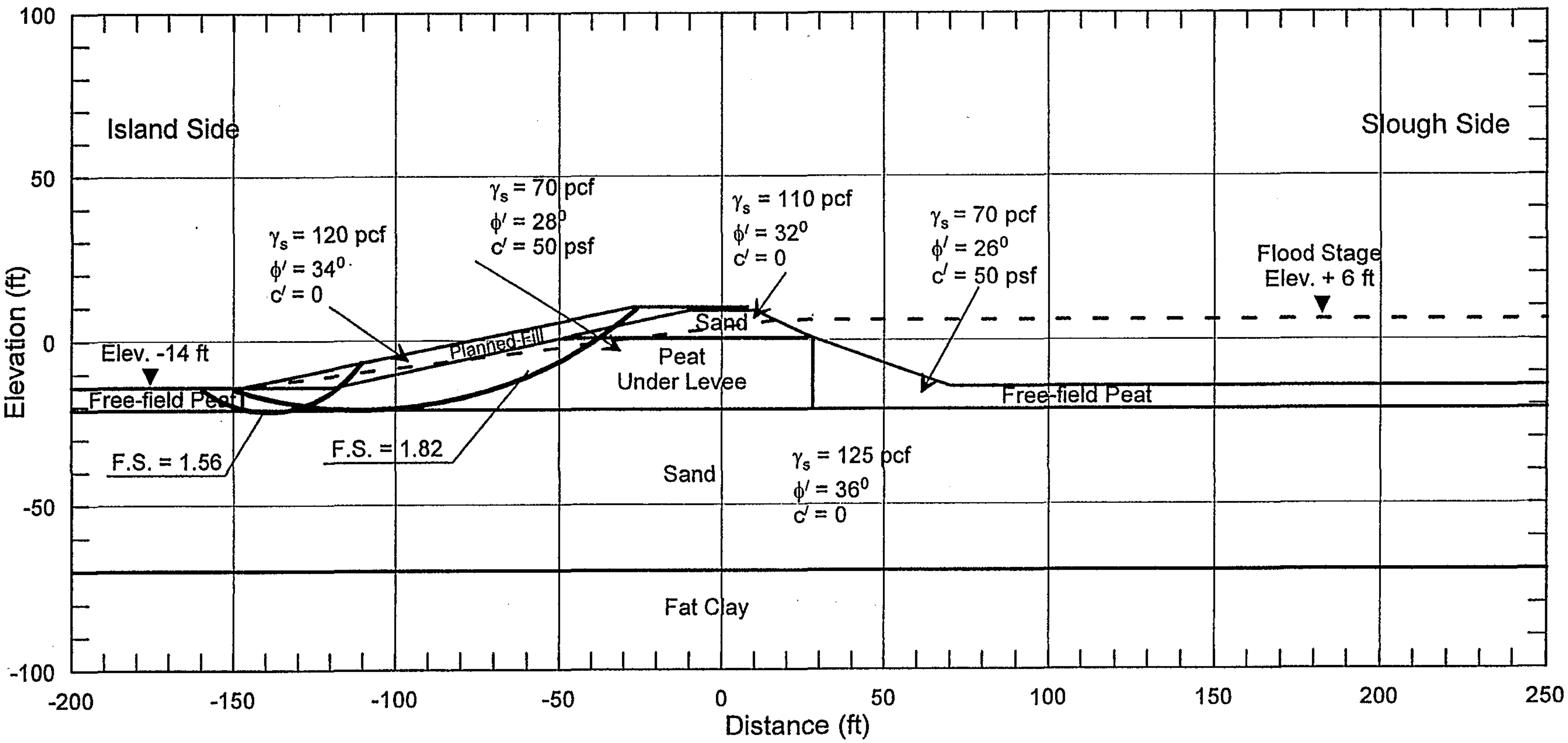

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS LONG-TERM CONDITION- INTO ISLAND	FIGURE 3.5.5
URS GREINER WOODWARD CLYDE		

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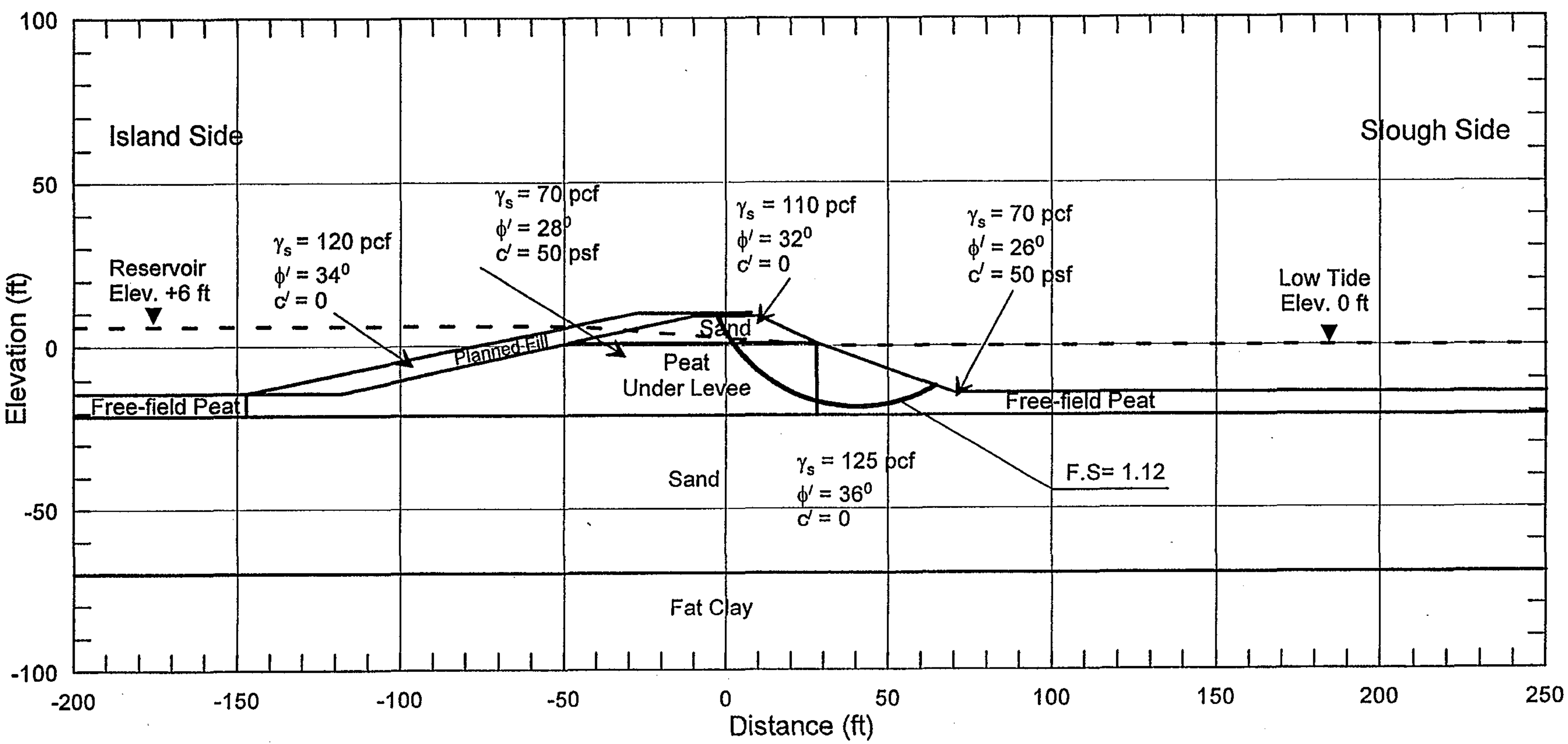

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	WEBB TRACT STA. 630+00 STABILITY ANALYSIS LONG-TERM CONDITION- TOWARD SLOUGH	FIGURE 3.5.6

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C — 0 6 3 4 7 7

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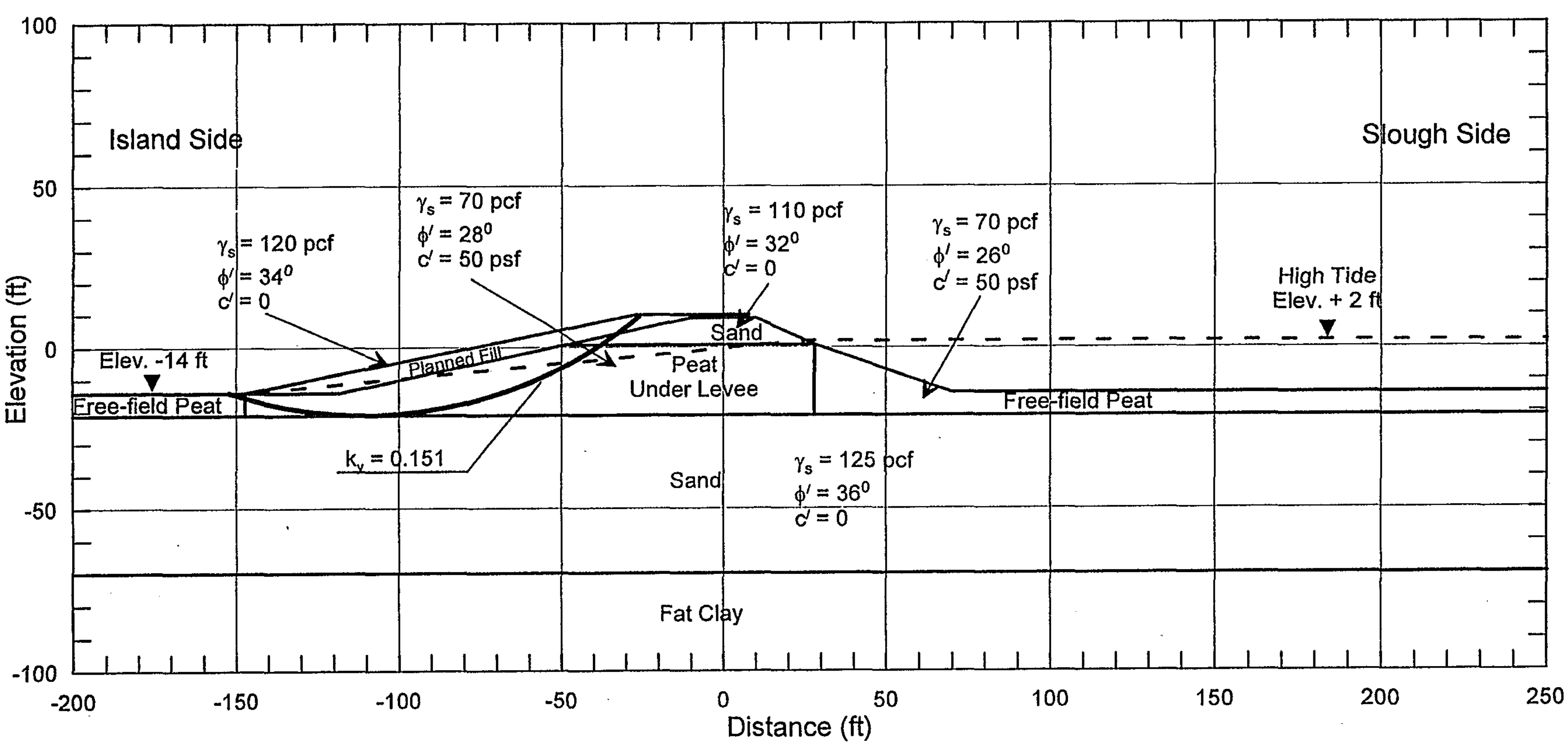

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS SEISMIC CONDITION- INTO ISLAND	FIGURE 3.5.7
URS GREINER WOODWARD CLYDE		

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C-063478

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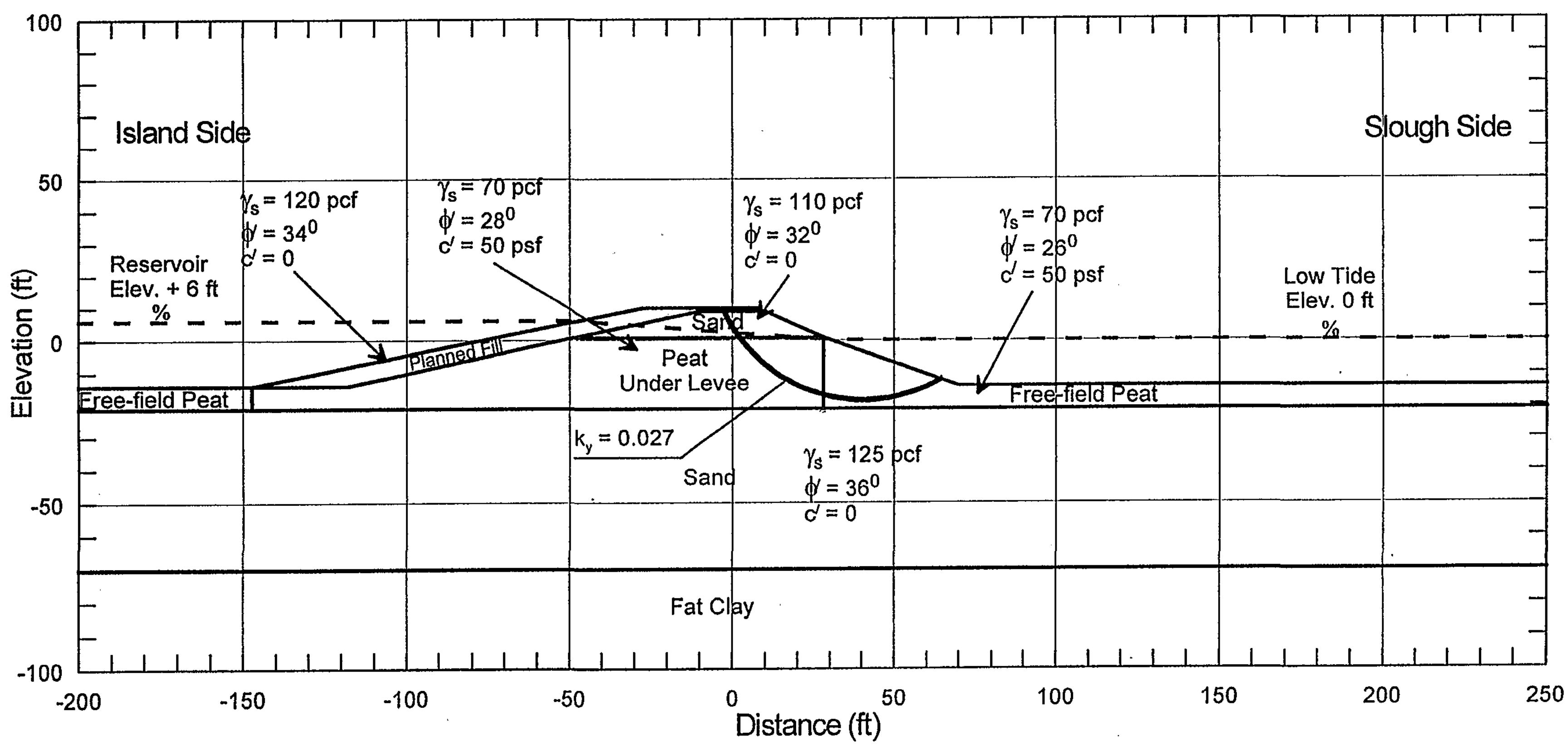

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS SEISMIC CONDITION TOWARD SLOUGH	FIGURE 3.5.8
URS Greiner Woodward Clyde		

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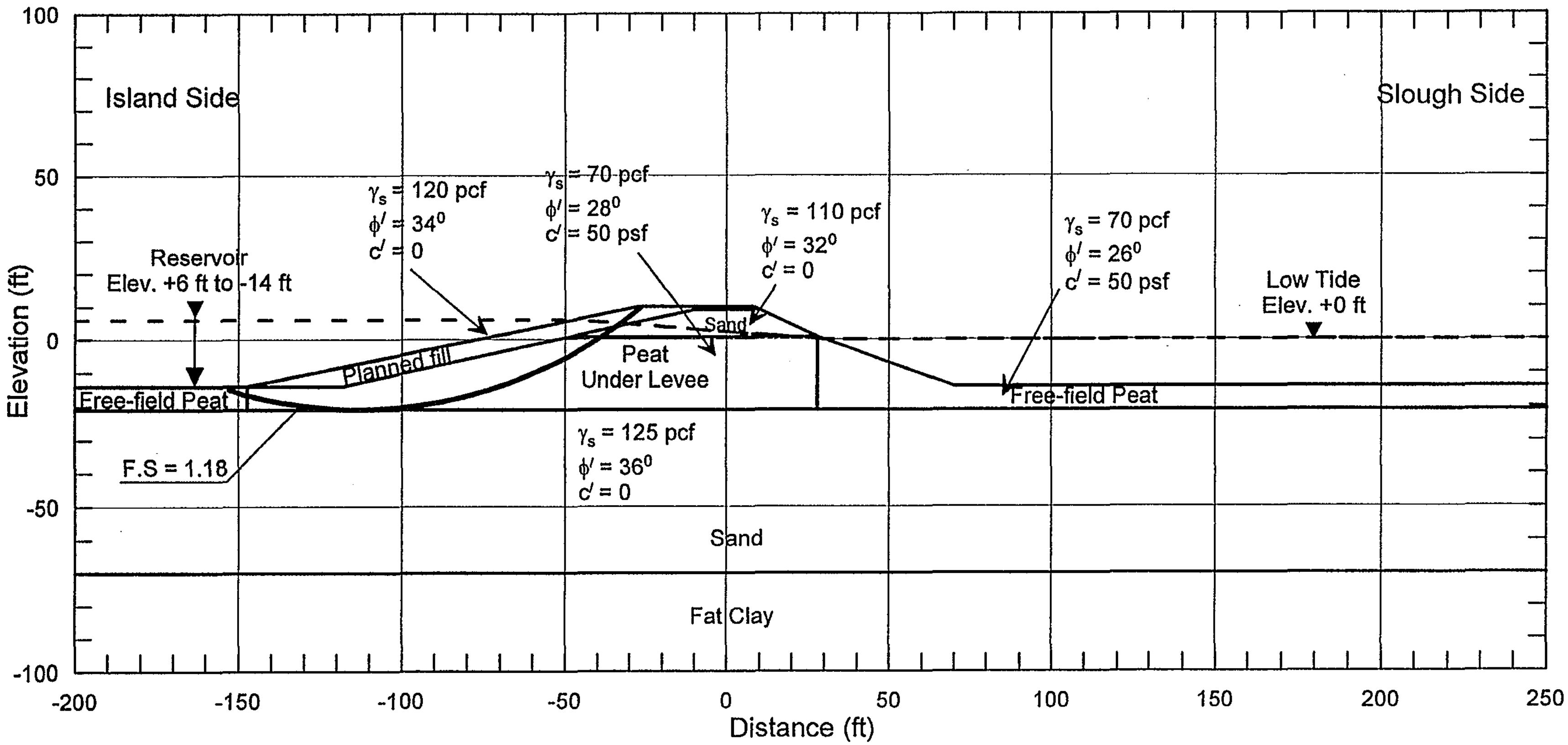

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS, THREE-STAGE SUDDEN DRAWDOWN CONDITION - INTO ISLAND	FIGURE 3.5.9
URS GREINER WOODWARD CLYDE		

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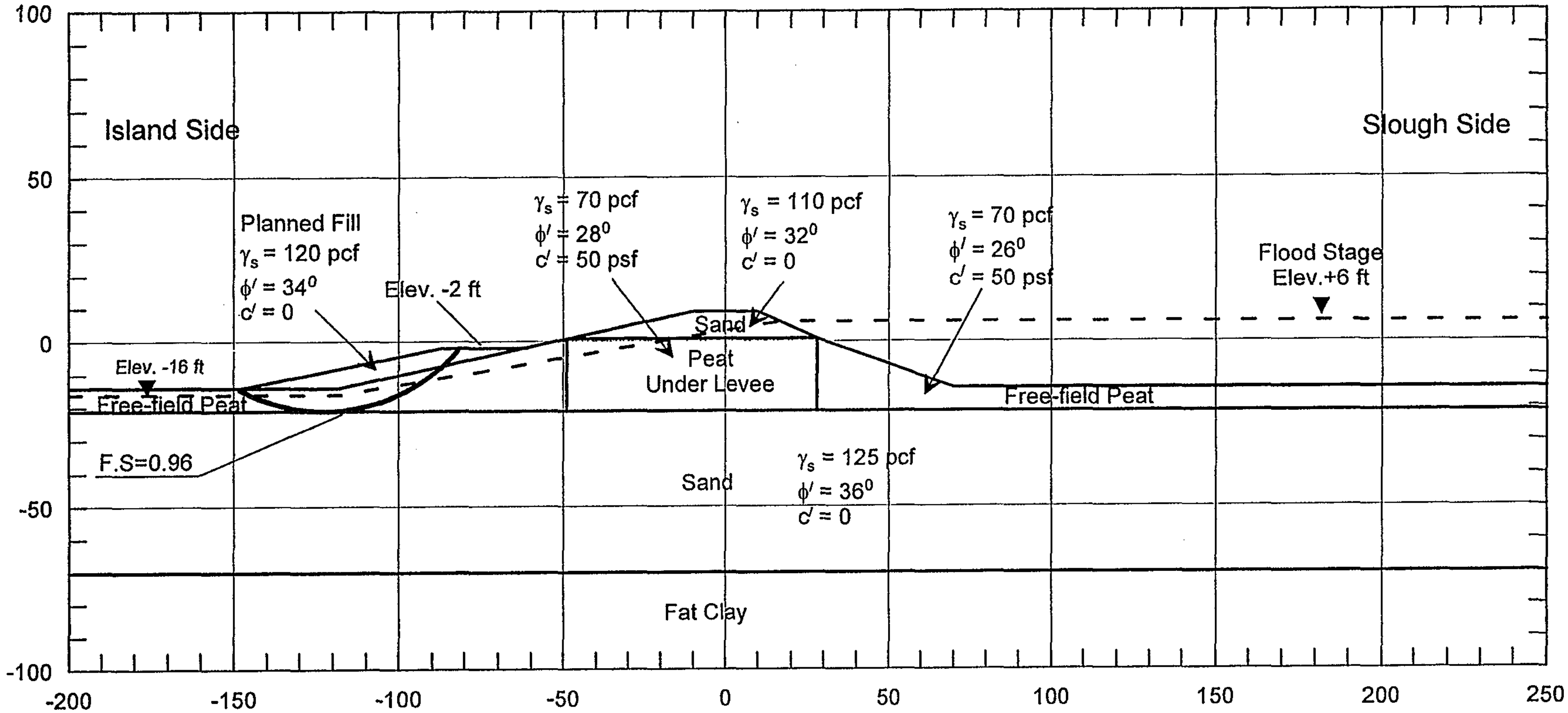
C-063480



DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS, TWO-STAGE END OF FIRST STAGE CONSTRUCTION - INTO ISLAND	FIGURE 3.5.10
URS GREINER WOODWARD CLYDE		

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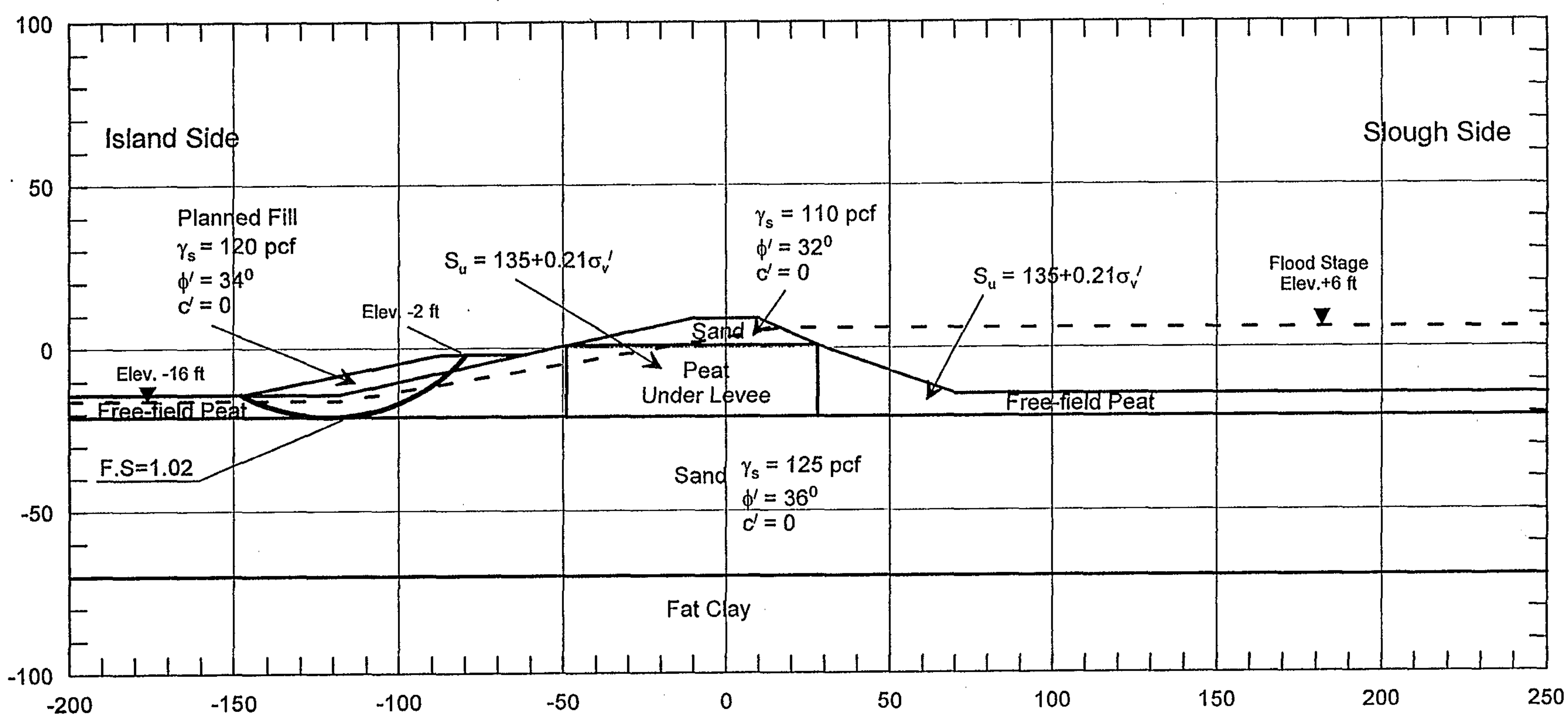

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	WEBB TRACT STA. 630+00 STABILITY ANALYSIS, Su PROFILE END OF FIRST STAGE CONSTRUCTION - INTO ISLAND	FIGURE 3.5.11

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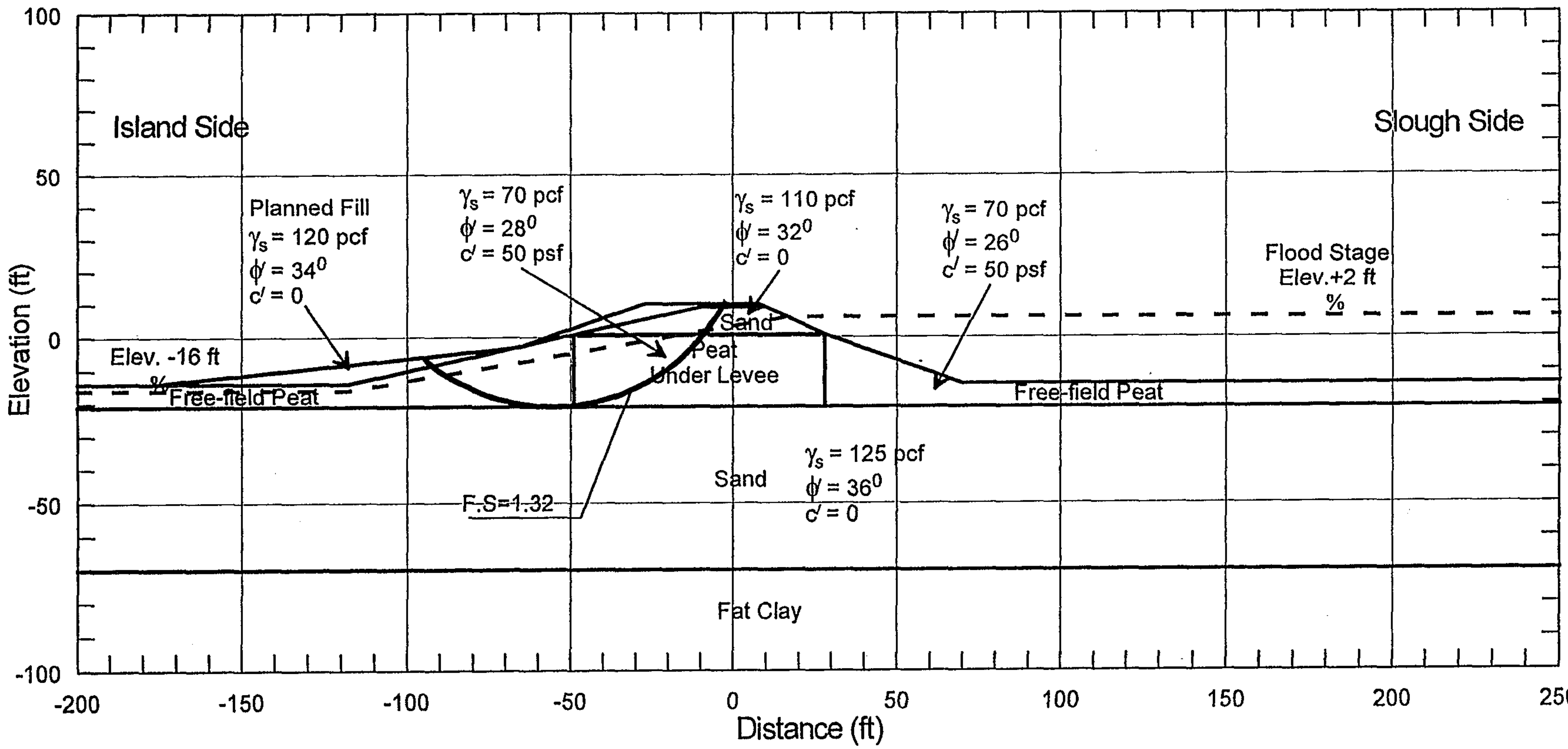

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	WEBB TRACT STA. 630+00 ALTERNATIVE GEOMETRY STABILITY ANALYSIS, TWO-STAGE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.12

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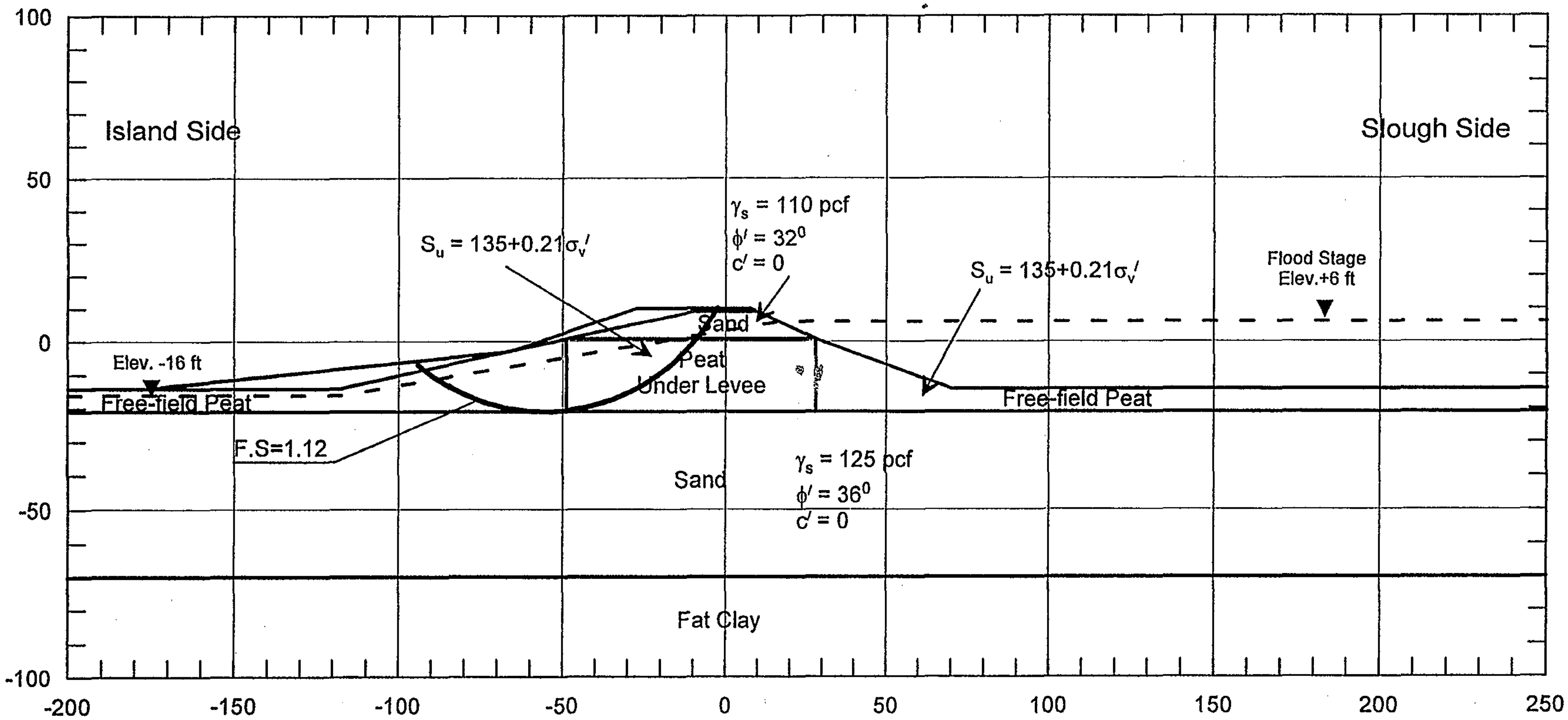

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS, S_u PROFILE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.13
URS GREINER WOODWARD CLYDE		

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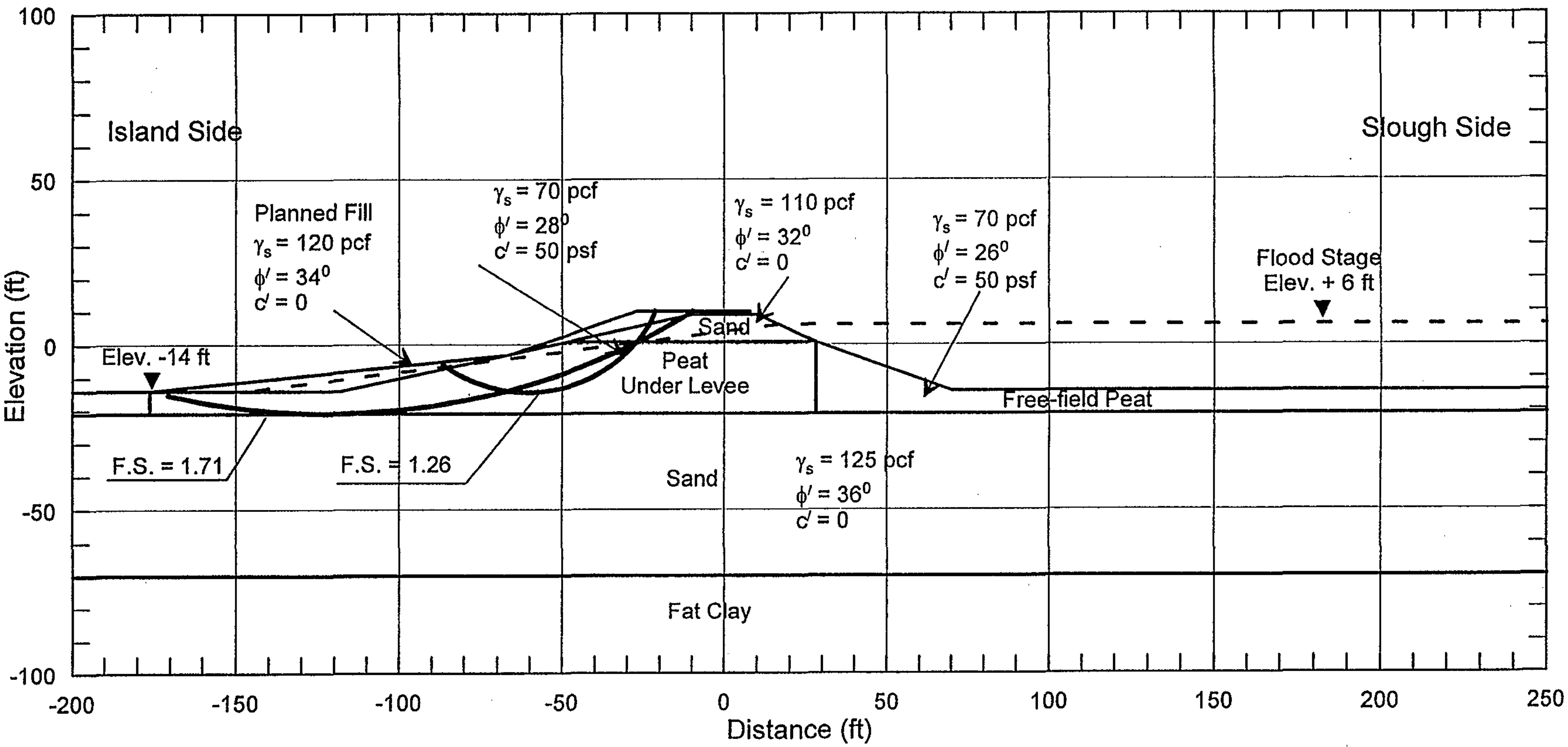

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS LONG-TERM CONDITION- INTO ISLAND	FIGURE 3.5.14
URS GREINER WOODWARD CLYDE		

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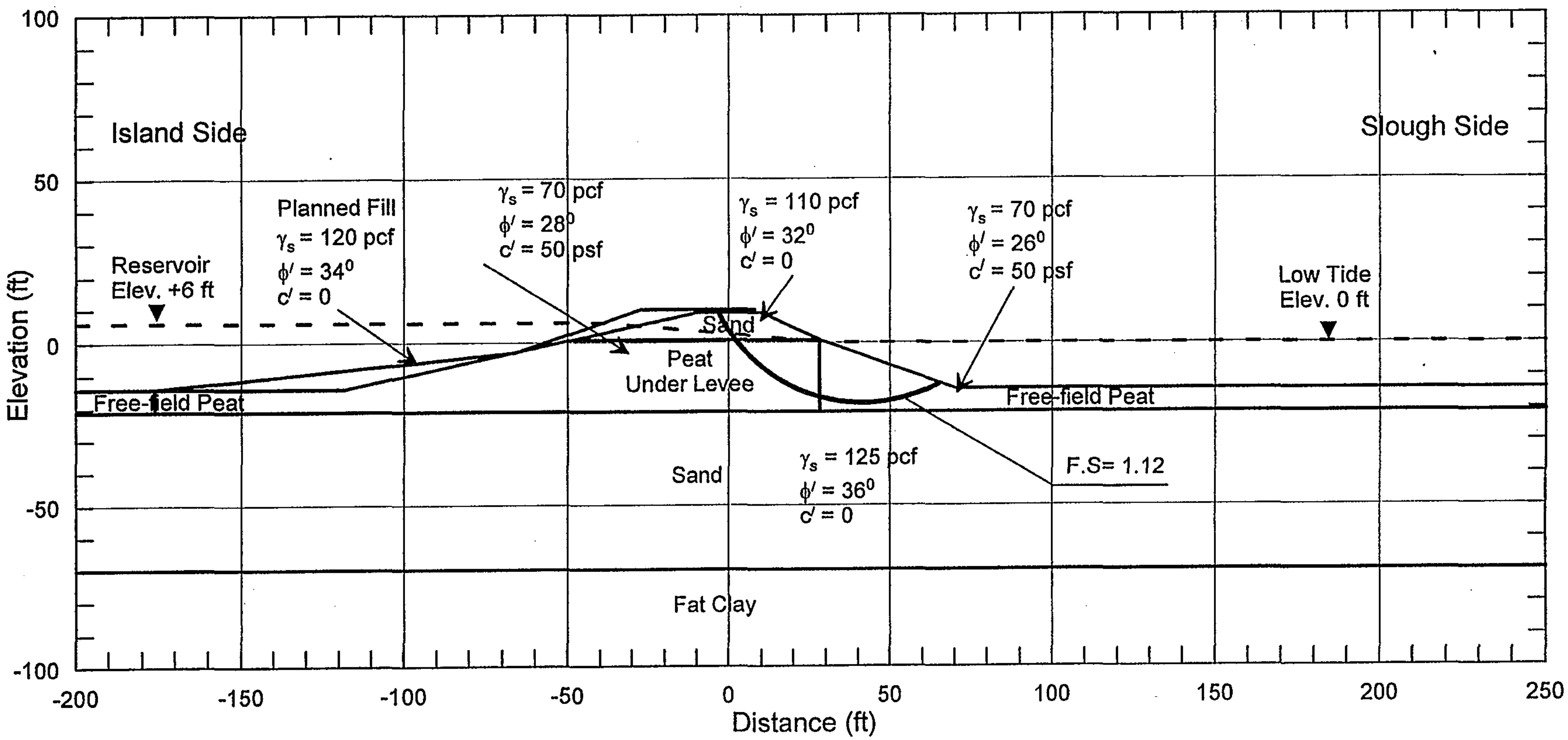

DELTA WETLANDS PROJECT	**WEBB TRACT STA. 630+00 STABILITY ANALYSIS LONG-TERM CONDITION- TOWARD SLOUGH**	**FIGURE 3.5.15**
URS GREINER WOODWARD CLYDE		

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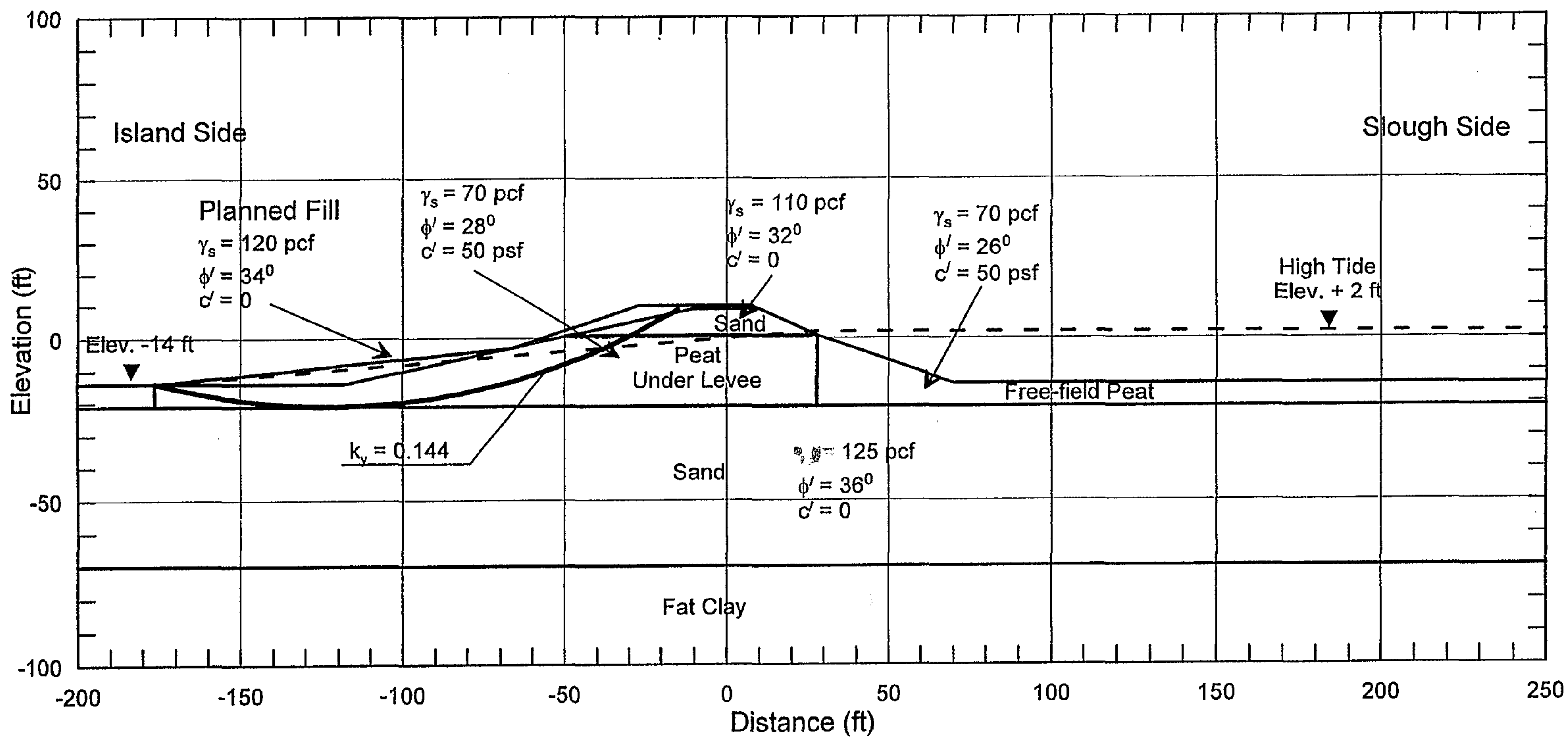

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	WEBB TRACT STA. 630+00 STABILITY ANALYSIS SEISMIC CONDITION- INTO ISLAND	FIGURE 3.5.16

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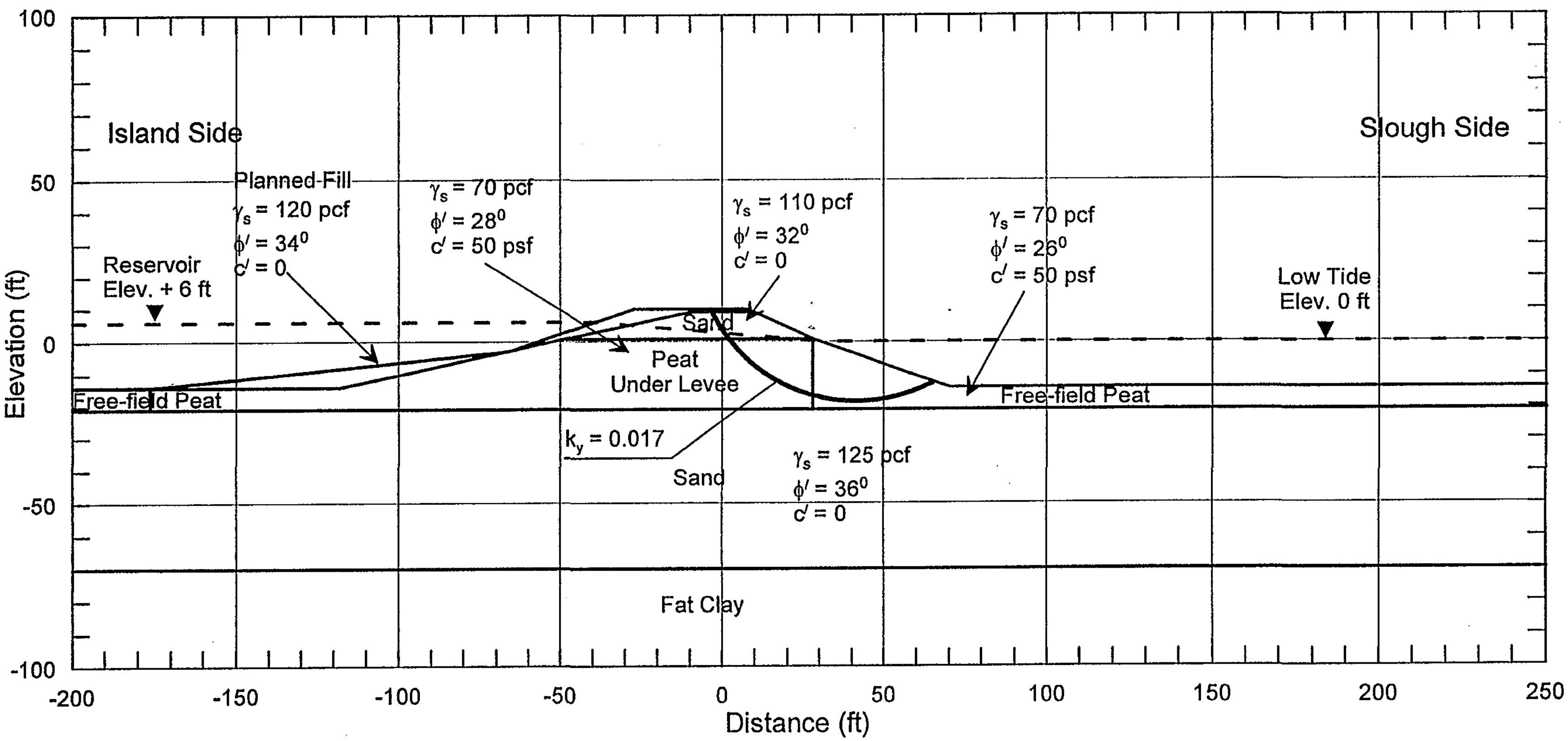

DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS SEISMIC CONITION TOWARD SLOUGH	FIGURE 3.5.17
URS Greiner Woodward Clyde		

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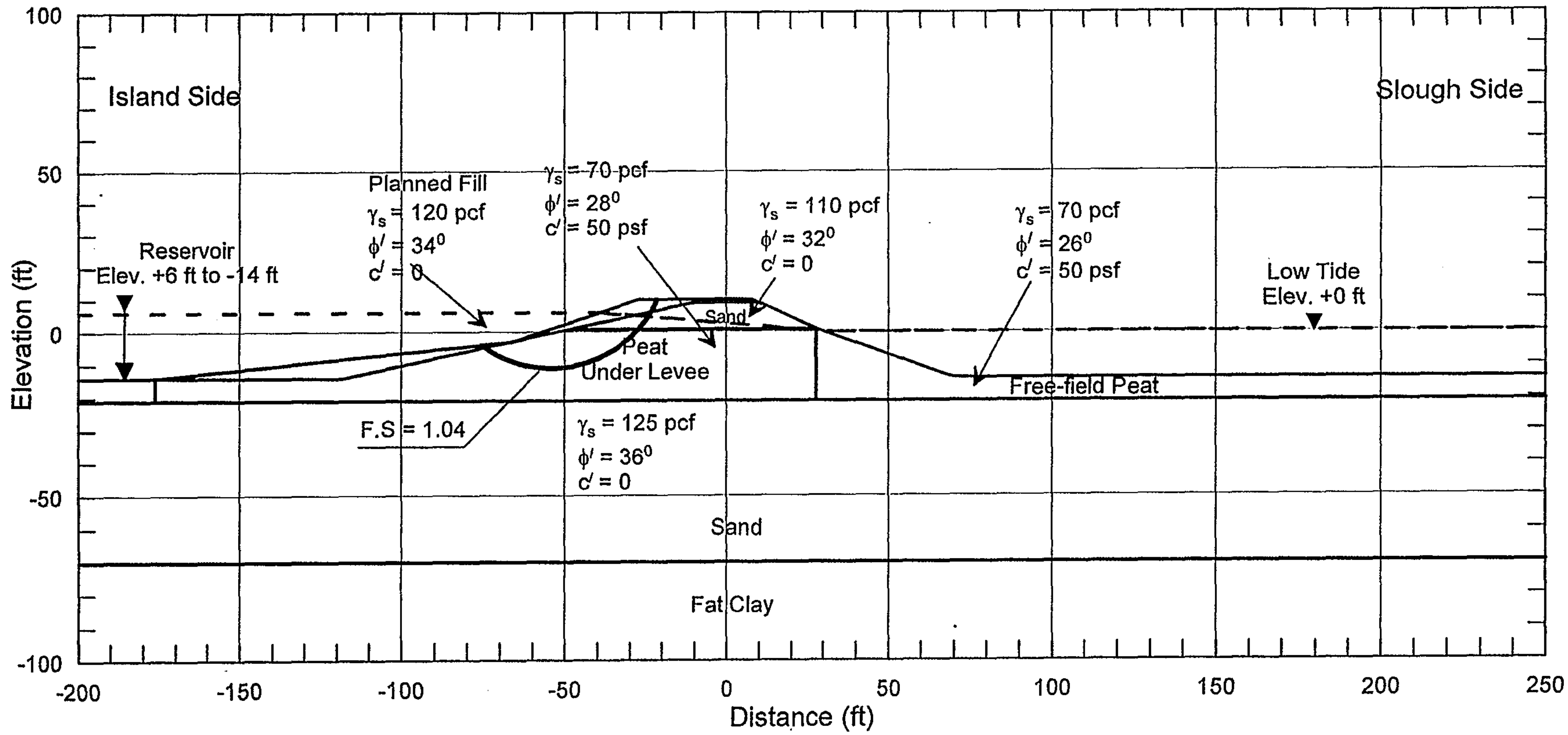
C-063489



DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	WEBB TRACT STA. 630+00 STABILITY ANALYSIS, THREE-STAGE SUDDEN DRAWDOWN CONDITION - INTO ISLAND	FIGURE 3.5.18

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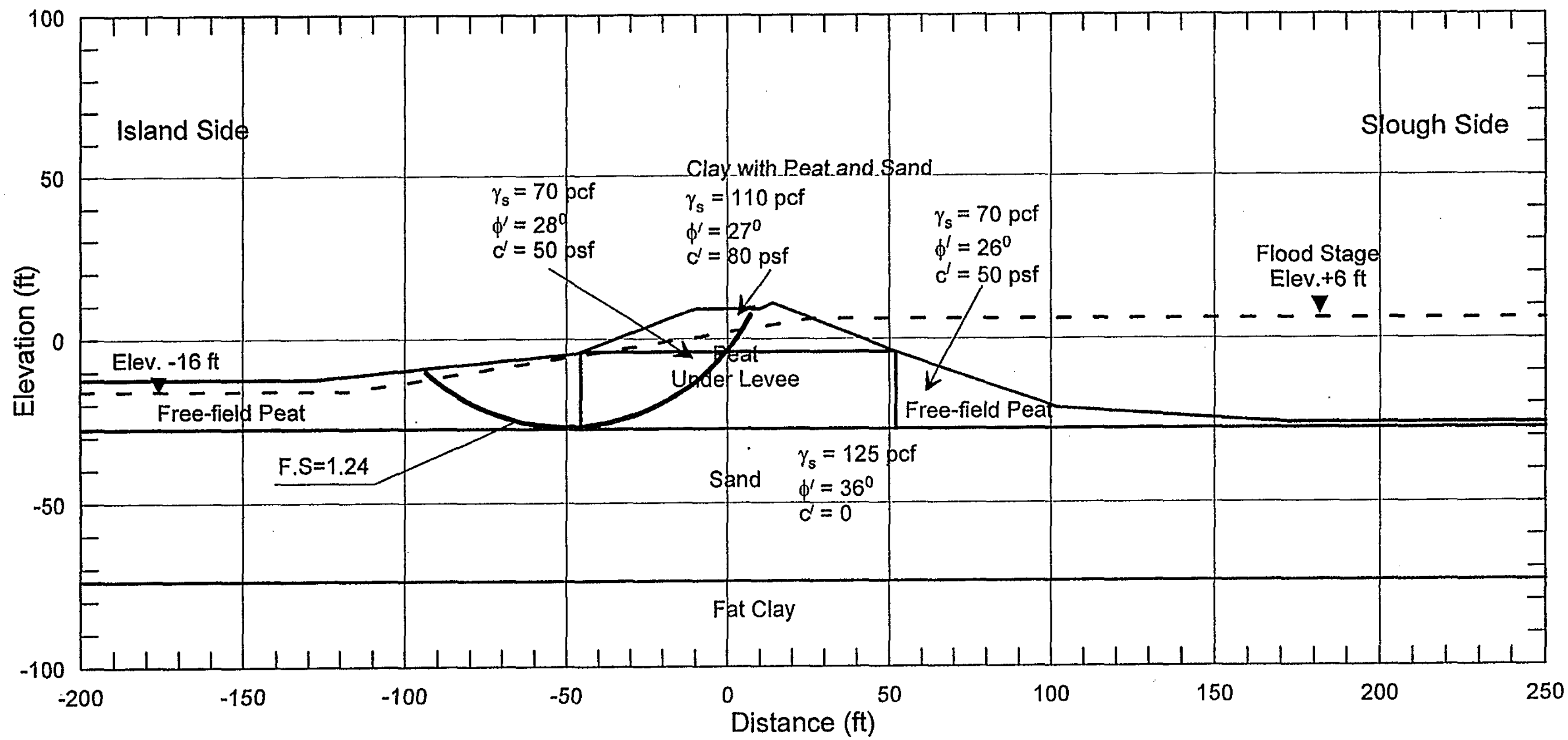

DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS EXISTING CONDITION-INTO ISLAND	FIGURE 3.5.19
URS GREINER WOODWARD CLYDE		

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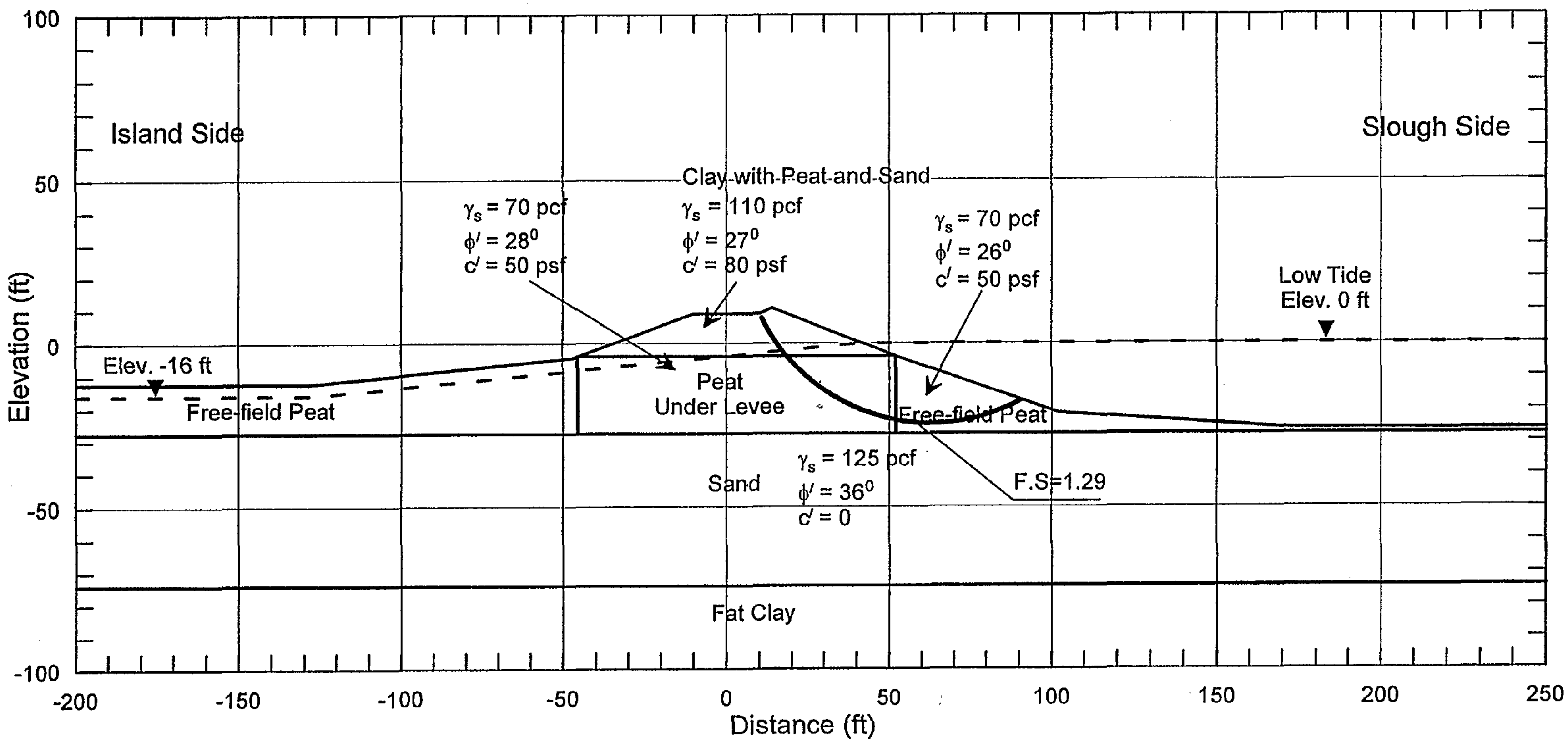

DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS EXISTING CONDITION- TOWARD SLOUGH	FIGURE 3.5.20
URS GREINER WOODWARD CLYDE		

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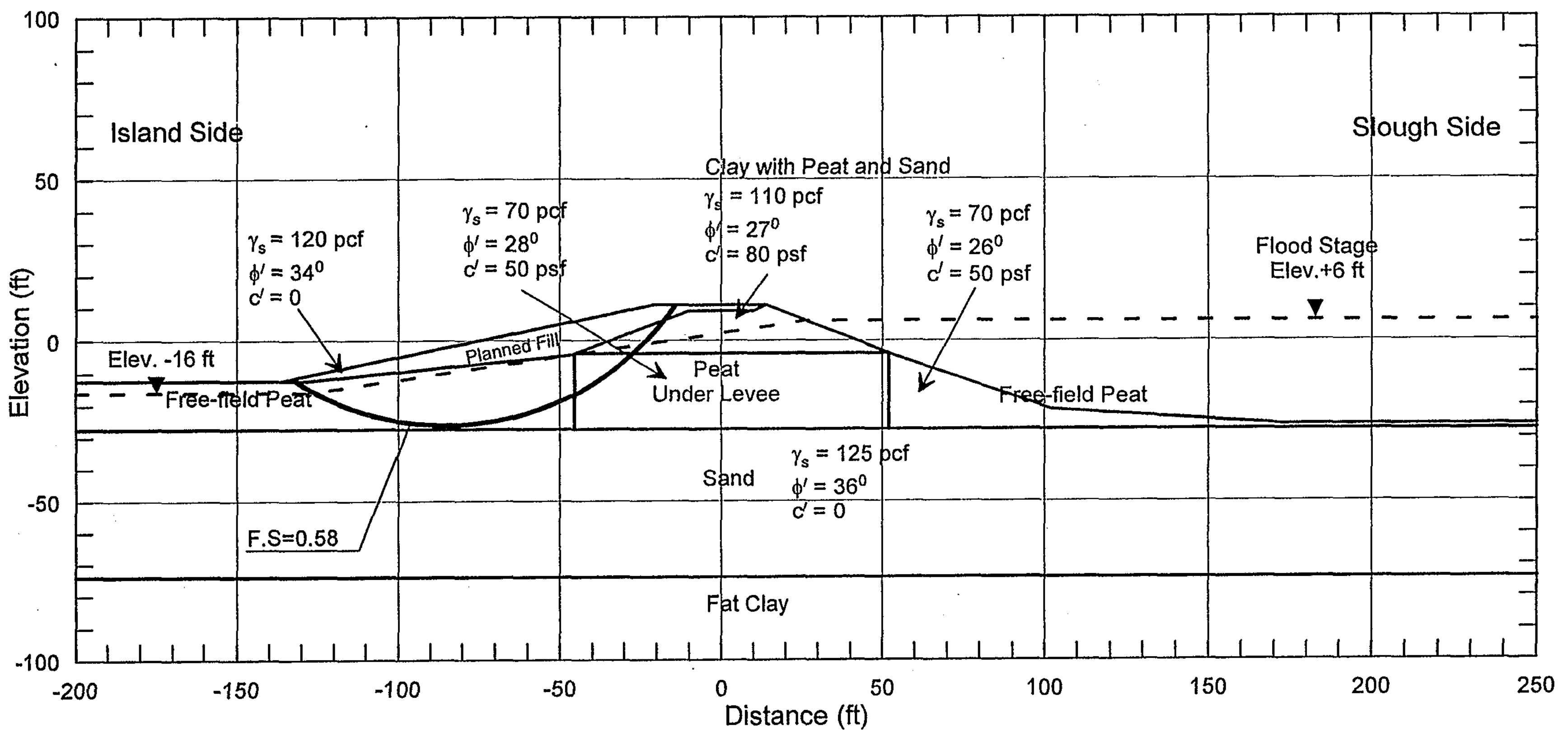
C-063492



DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS, TWO-STAGE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.21
URS GREINER WOODWARD CLYDE		

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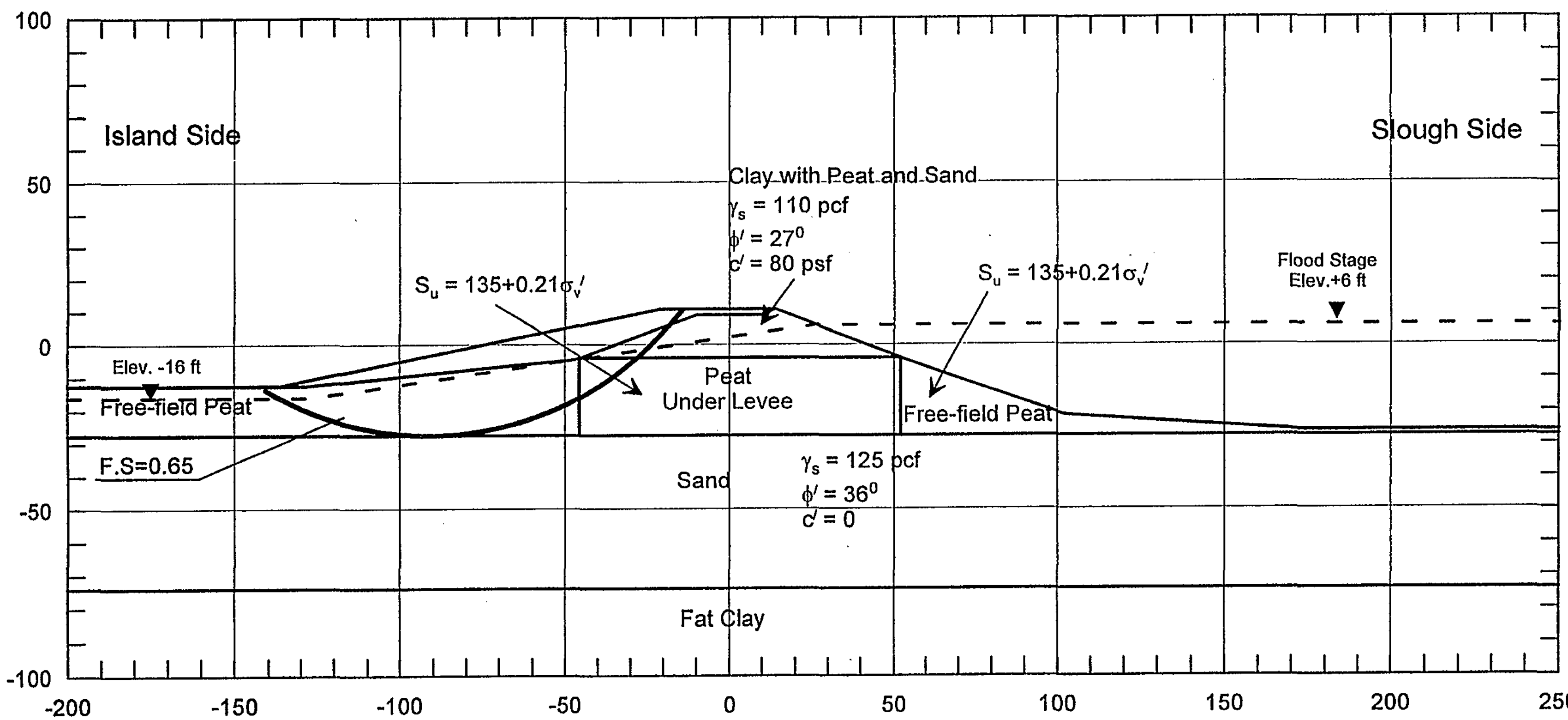
C-063493



DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS, S_u PROFILE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.22
URS GREINER WOODWARD CLYDE		

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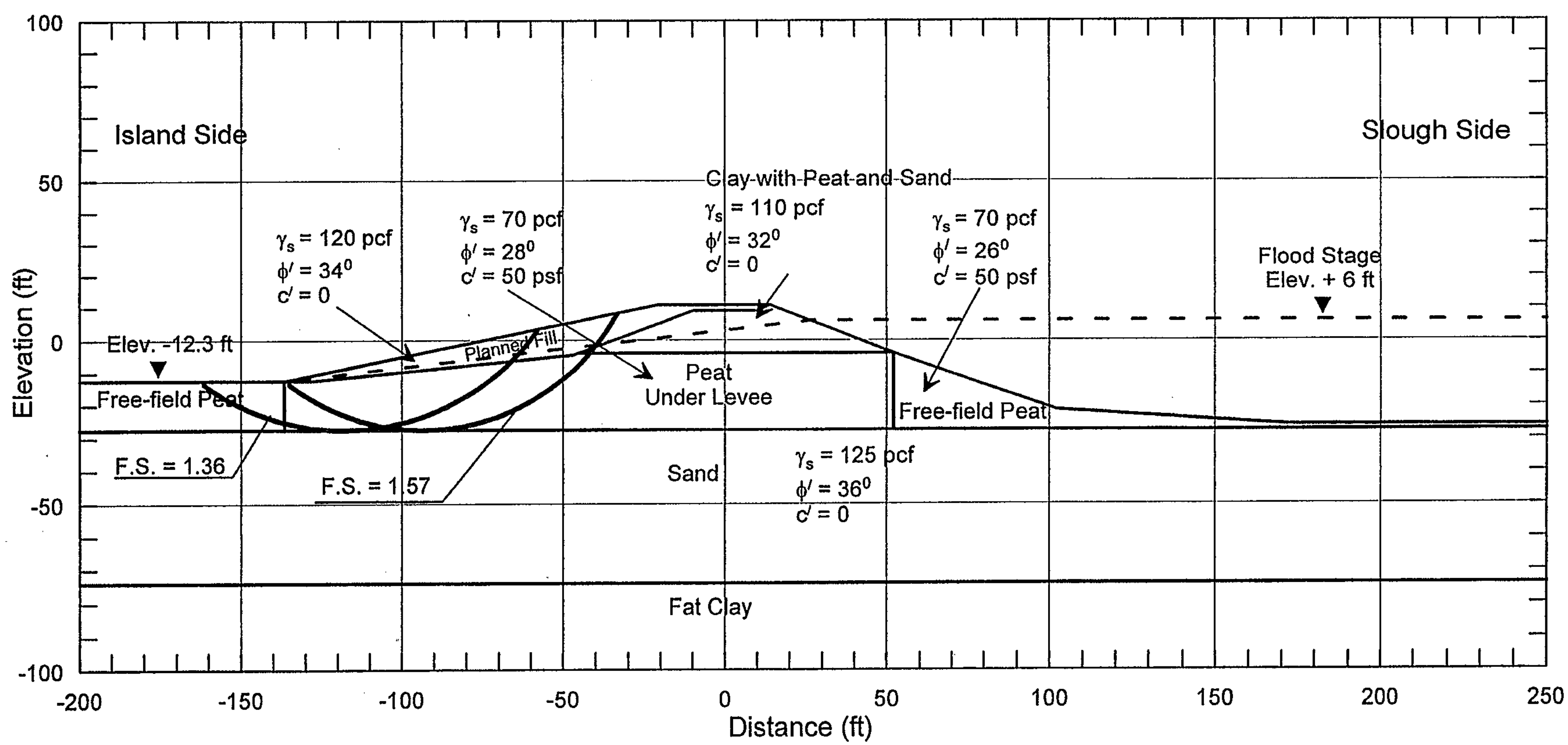
C-063493



DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS LONG-TERM CONDITION- INTO ISLAND	FIGURE 3.5.23
URS GREINER WOODWARD CLYDE		

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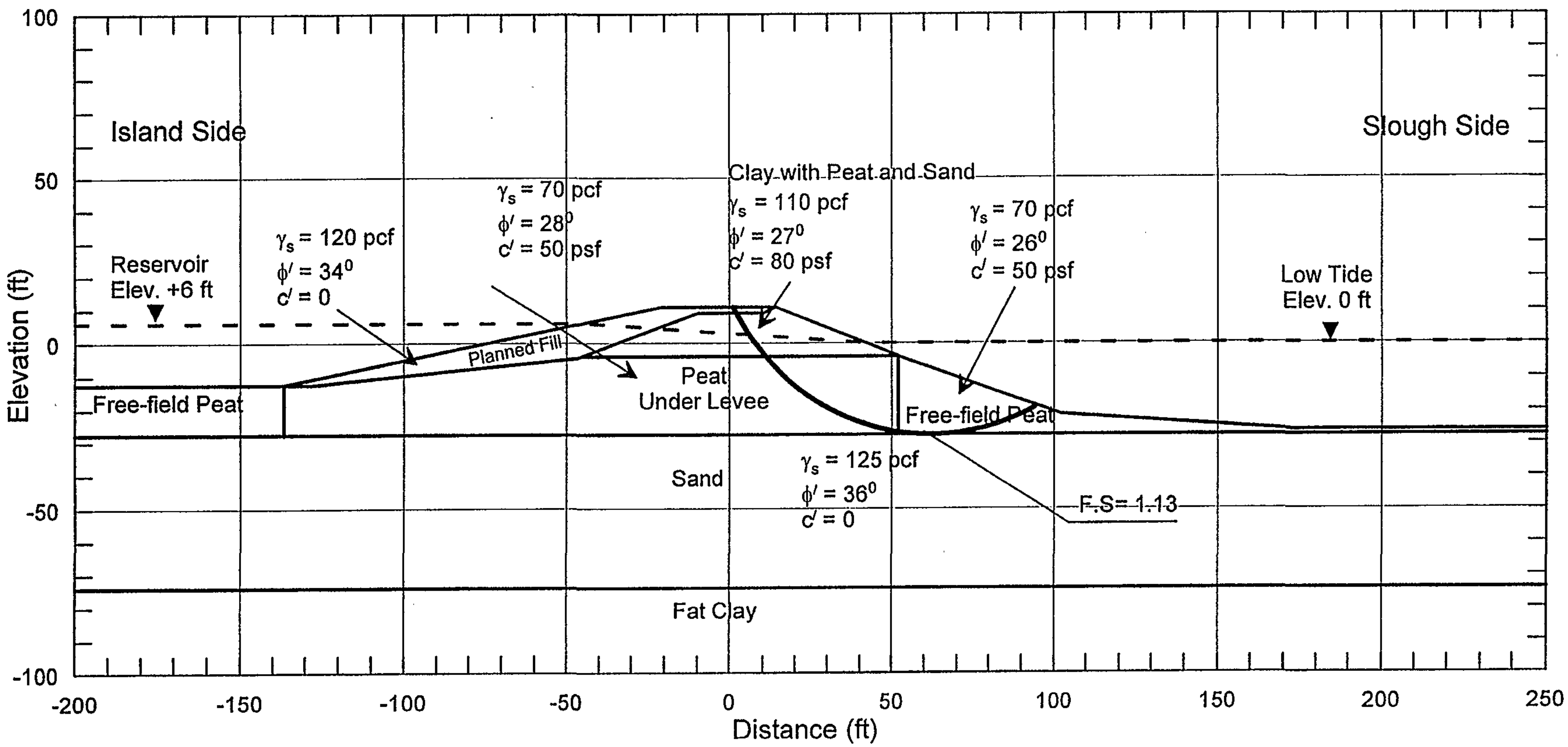
C-063495



DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS LONG-TERM CONDITION- TOWARD SLOUGH	FIGURE 3.5.24
URS GREINER WOODWARD CLYDE		

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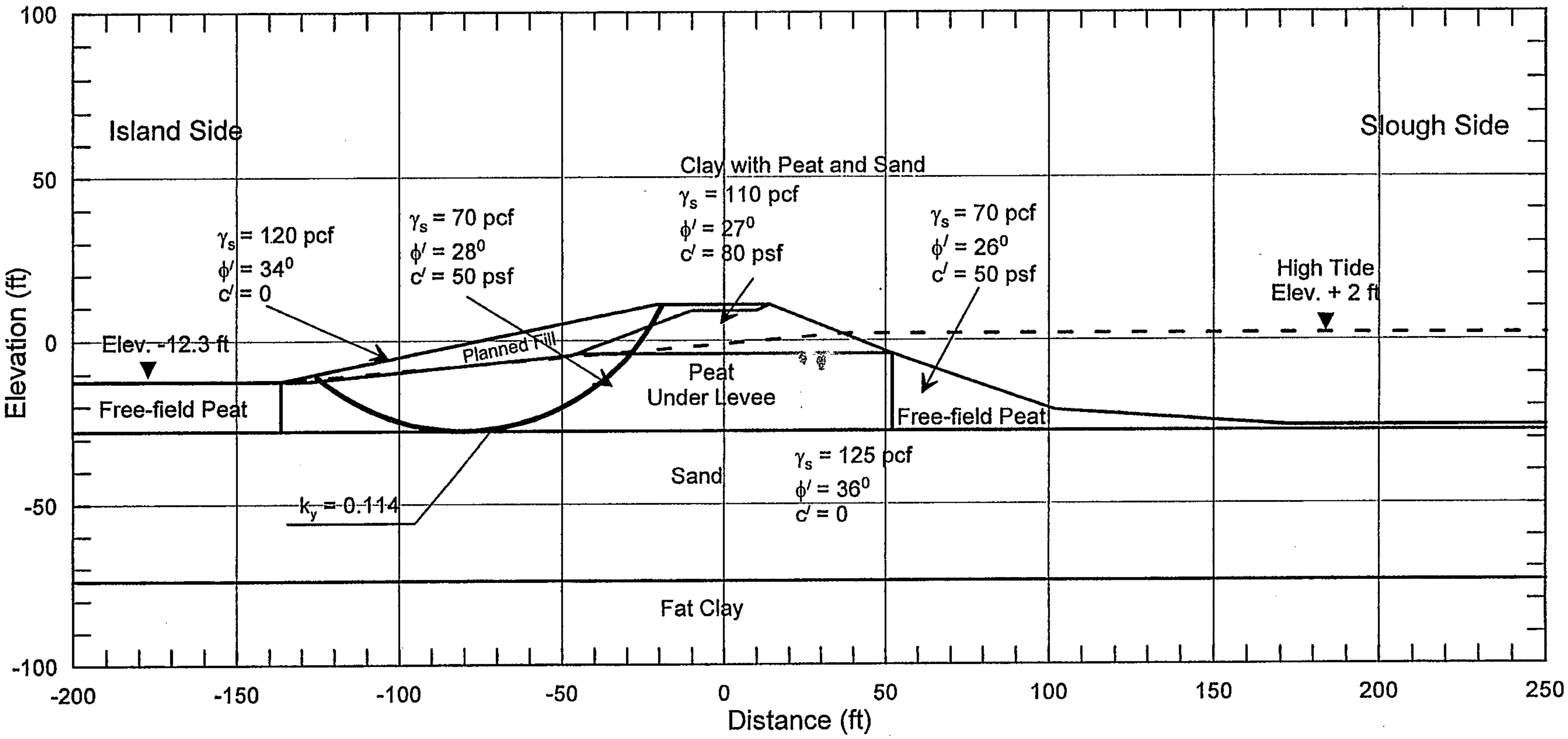
C—063495



DELTA WETLANDS PROJECT	**WEBB TRACT STA. 160+00 STABILITY ANALYSIS SEISMIC CONDITION- INTO ISLAND**	**FIGURE 3.5.25**
URS GREINER WOODWARD CLYDE		

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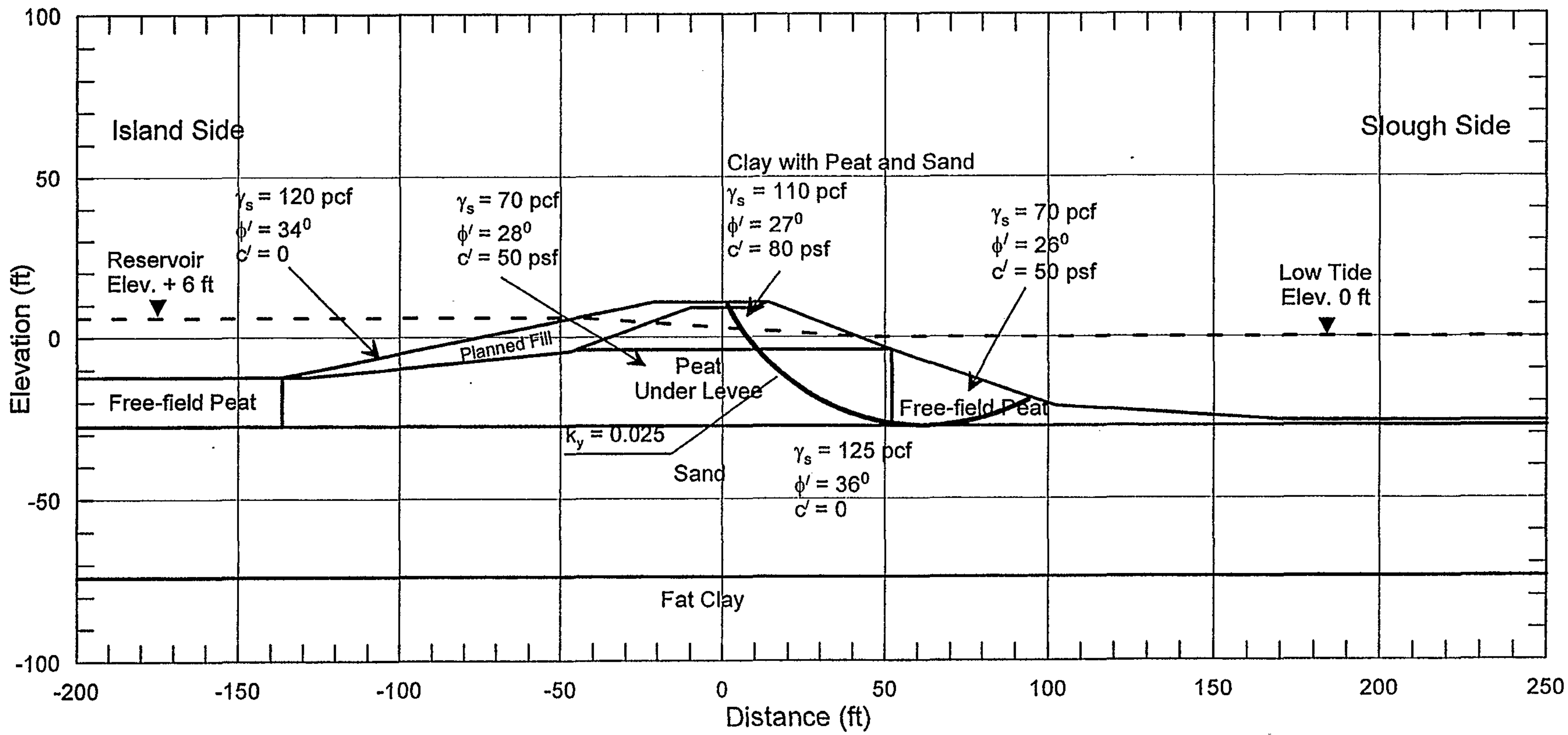
C-063497



DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS SEISMIC CONITION TOWARD SLOUGH	FIGURE 3.5.26
URS Greiner Woodward Clyde		

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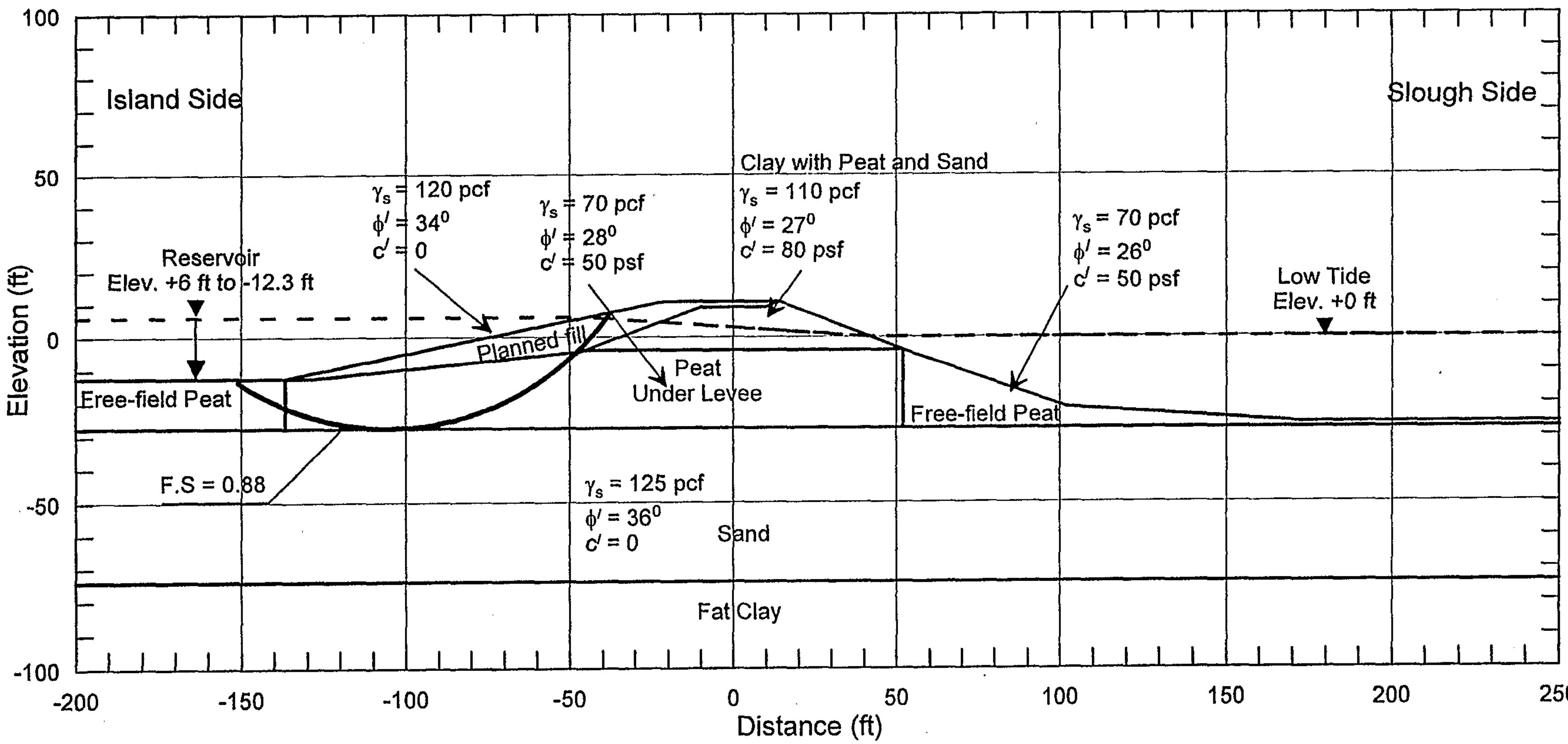
C-063497



DELTA WETLANDS PROJECT	WEBB TRACT STA. 160+00 STABILITY ANALYSIS, THREE-STAGE SUDDEN DRAWDOWN CONDITION - INTO ISLAND	FIGURE 3.5.27
URS GREINER WOODWARD CLYDE		

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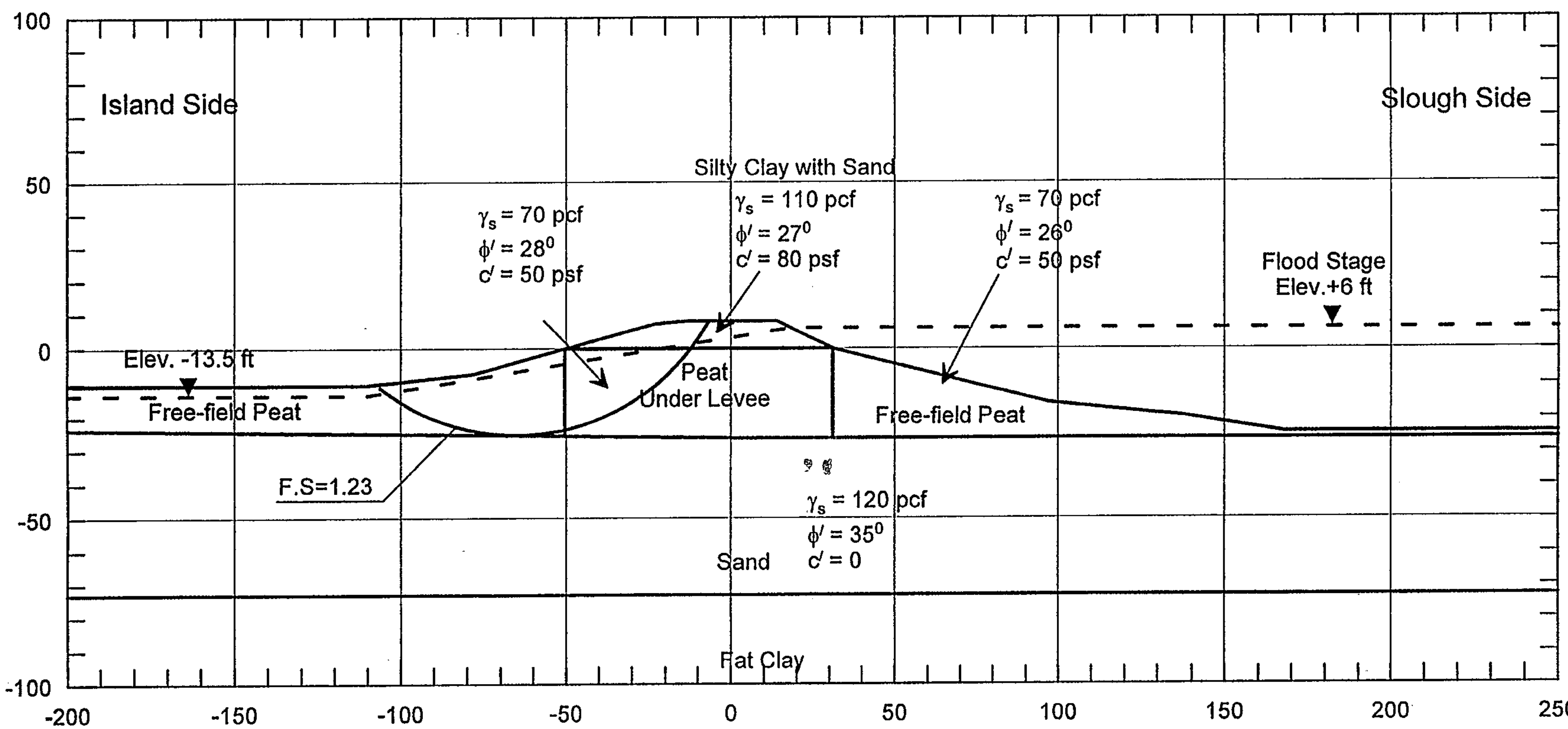

DELTA WETLANDS PROJECT *URS GREINER WOODWARD CLYDE*	BACON ISLAND STA. 25+00 STABILITY ANALYSIS EXISTING CONDITON- INTO ISLAND	FIGURE 3.5.28

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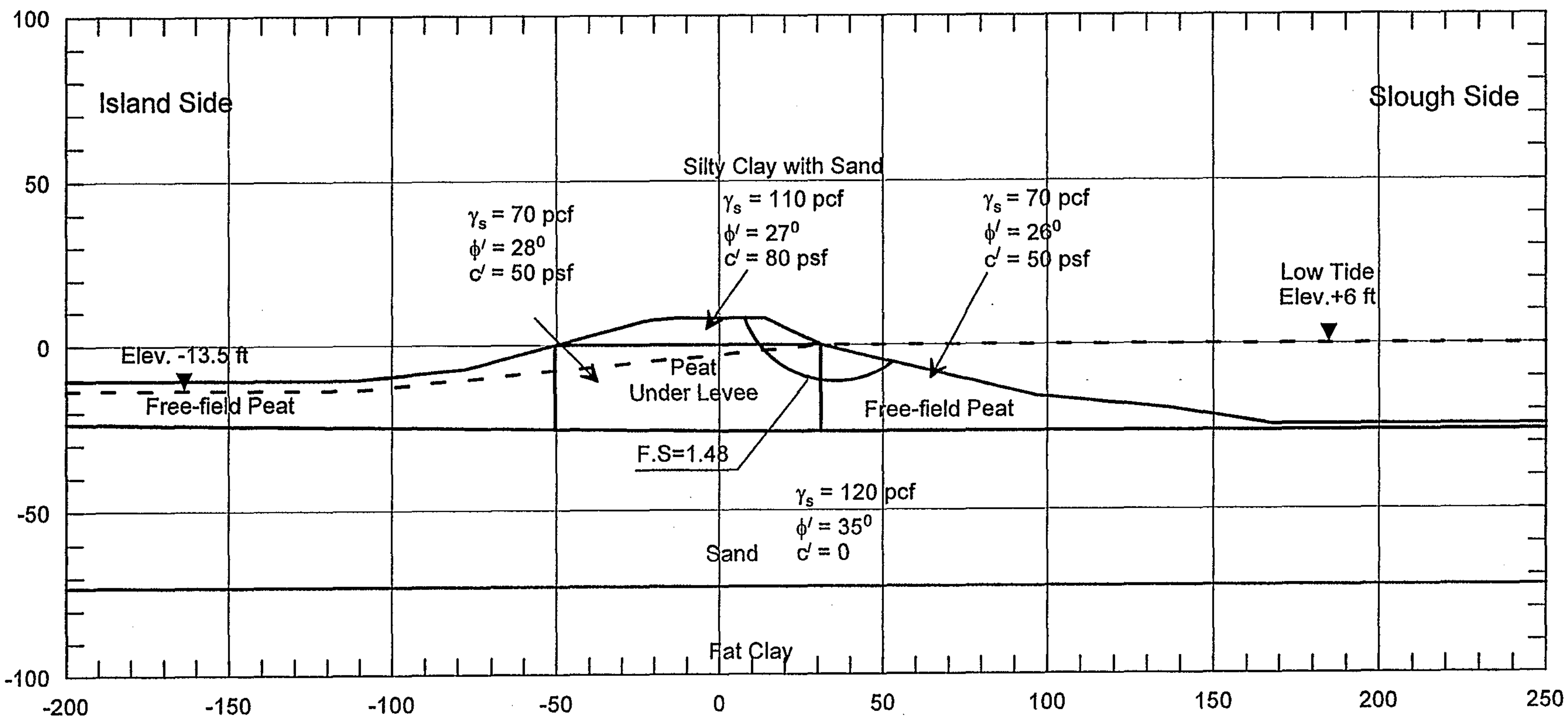
C — 0 6 3 4 9 9

C-063499



DELTA WETLANDS PROJECT	BACON ISLAND STA. 25+00 STABILITY ANALYSIS EXISTING CONDITON- TOWARD SLOUGH	FIGURE 3.5.29
URS GREINER WOODWARD CLYDE		

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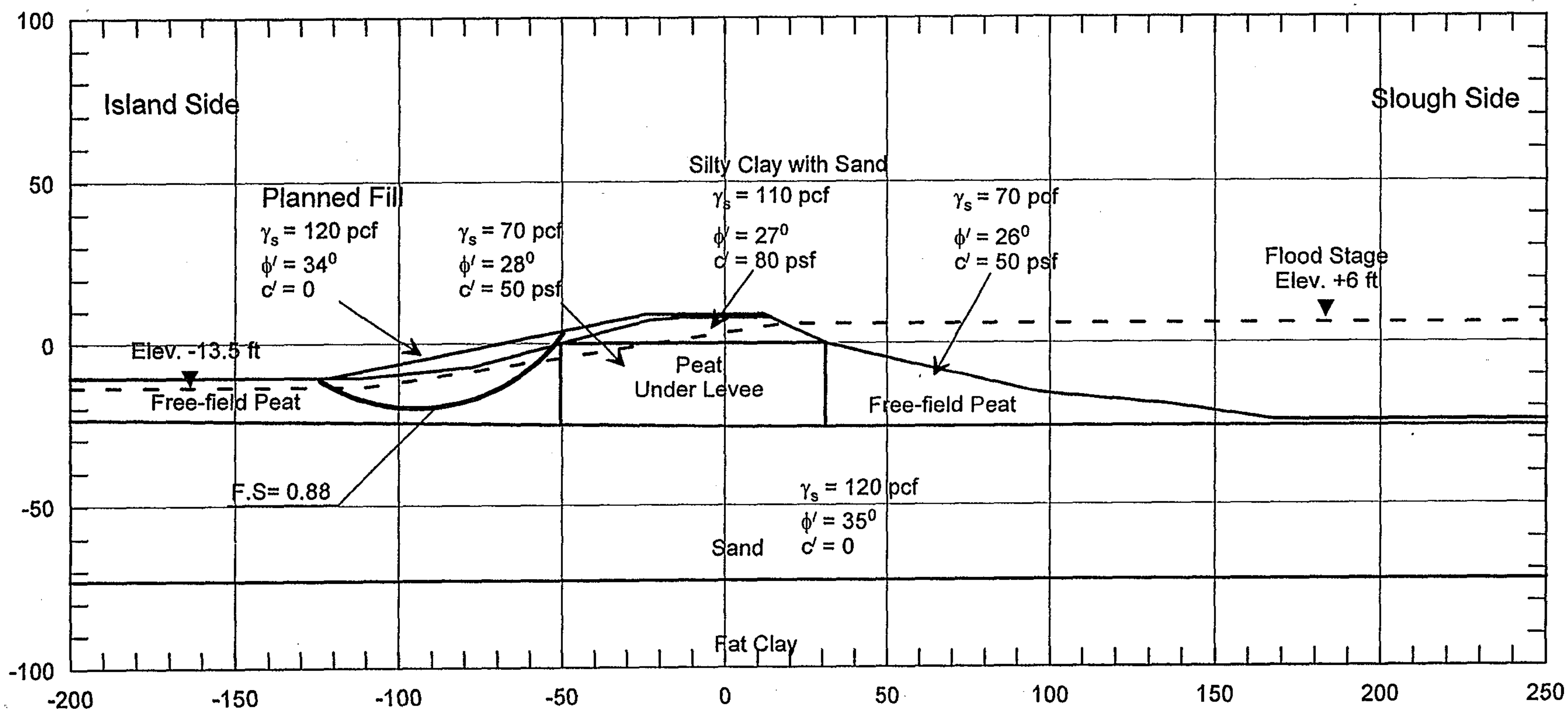

DELTA WETLANDS PROJECT	BACON ISLAND STA. 25+00 STABILITY ANALYSIS, TWO-STAGE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.30
URS GREINER WOODWARD CLYDE		

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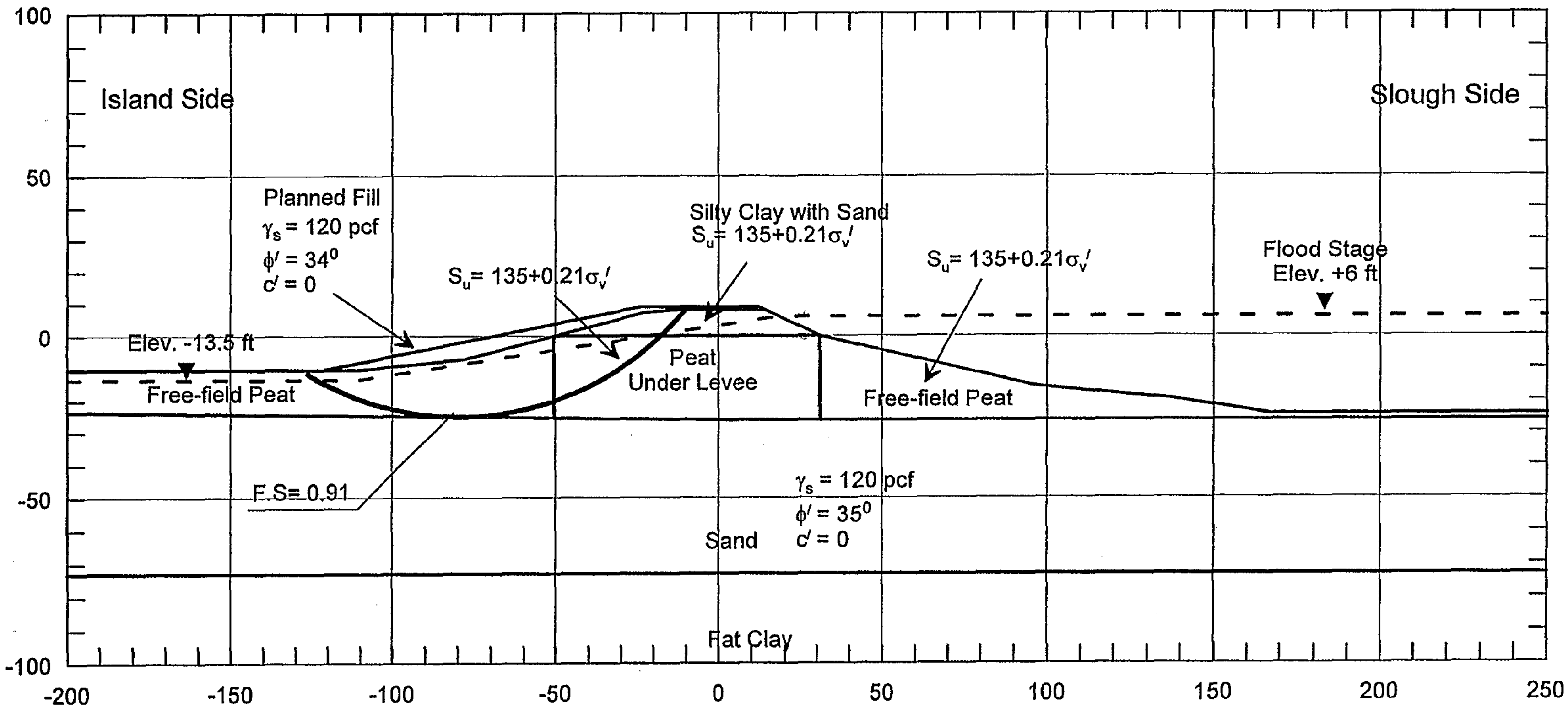
C-063502



DELTA WETLANDS PROJECT	**BACON ISLAND STA. 25+00 STABILITY ANALYSIS, S_u PROFILE END OF CONSTRUCTION- INTO ISLAND**	**FIGURE 3.5.31**
URS GREINER WOODWARD CLYDE		

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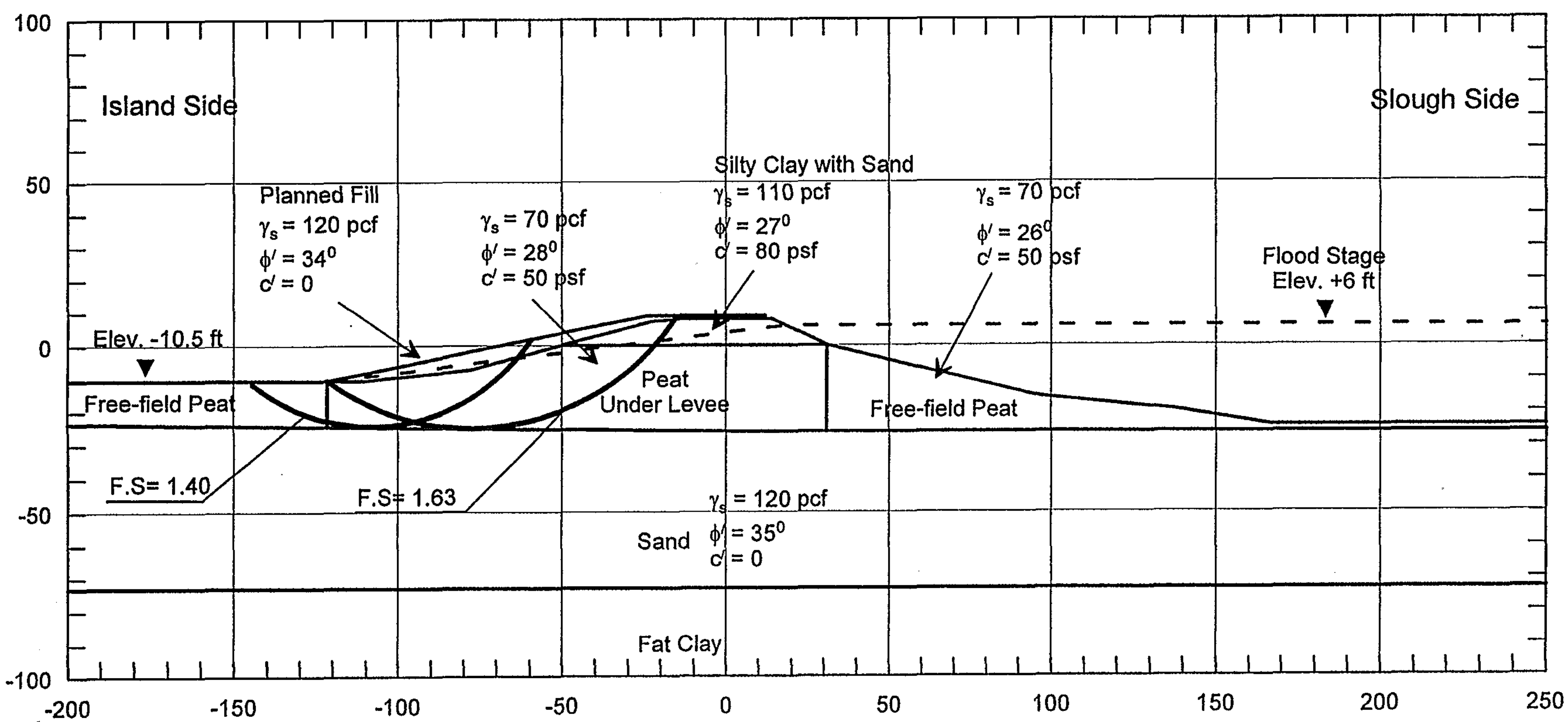
C-063502



DELTA WETLANDS PROJECT	BACON ISLAND STA. 25+00 STABILITY ANALYSIS LONG-TERM CONDITION- INTO ISLAND	FIGURE 3.5.32
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon25/ba25long.grf 12/2/99

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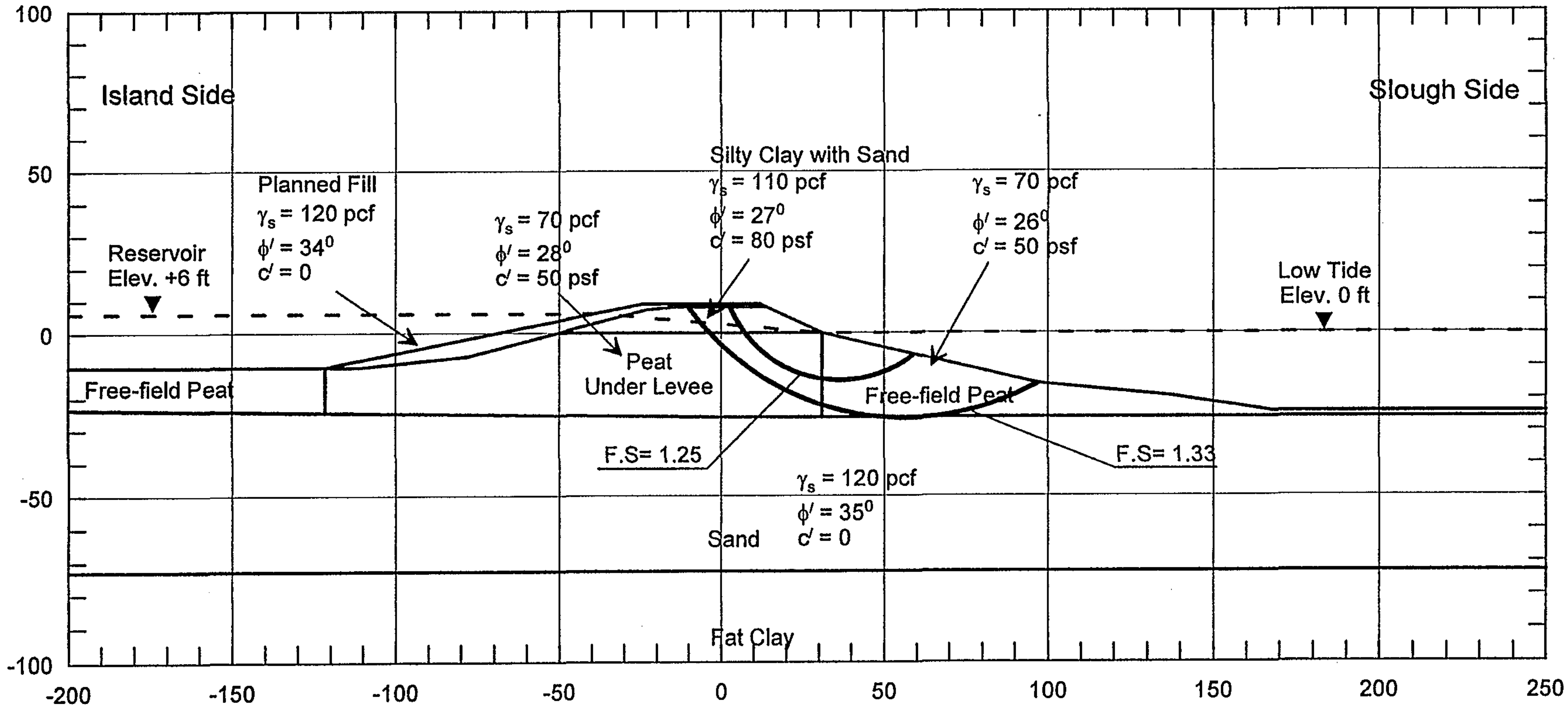
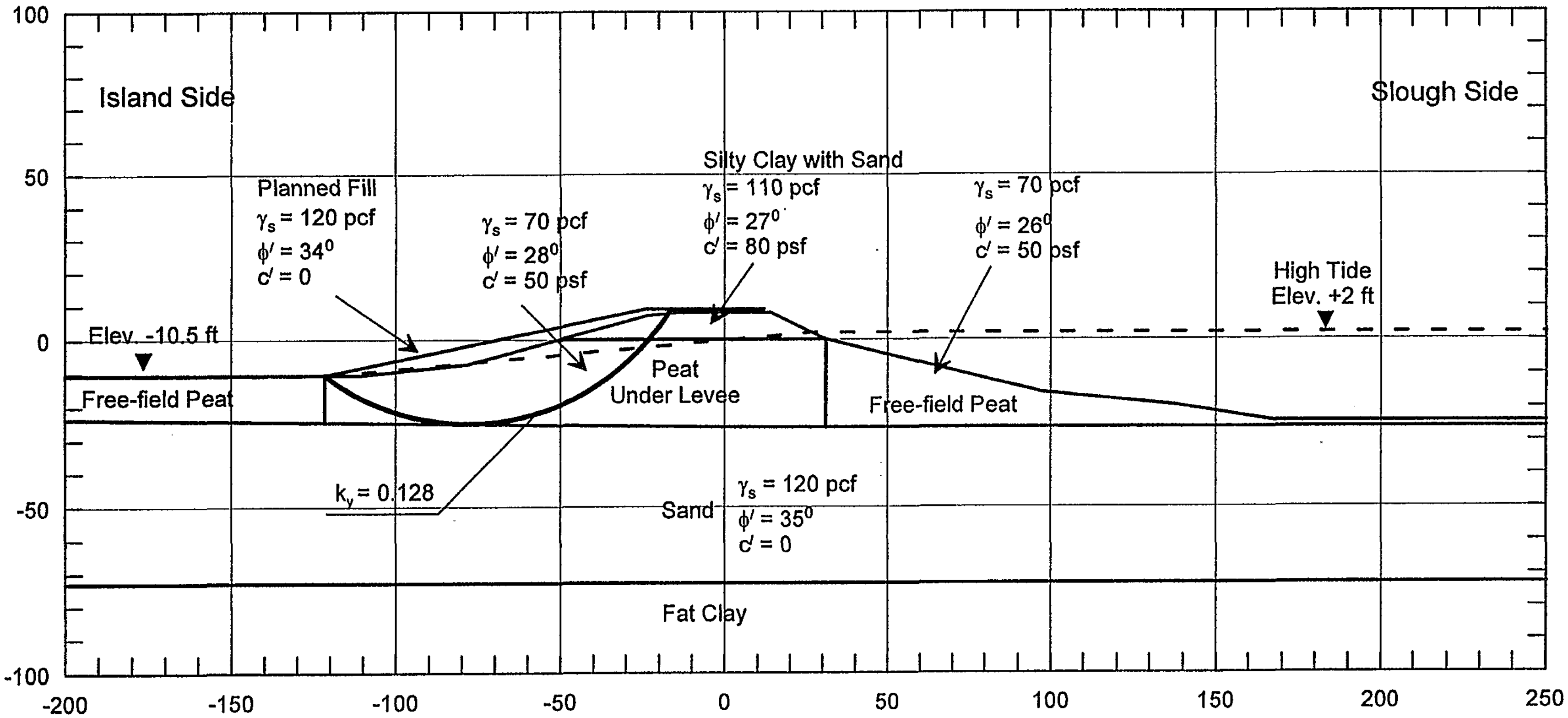
C-063504



DELTA WETLANDS PROJECT	BACON ISLAND STA. 25+00 STABILITY ANALYSIS LONG-TERM CONDITION- TOWARD SLOUGH	FIGURE 3.5.33
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon25/ba25lons.grf 12/2/99

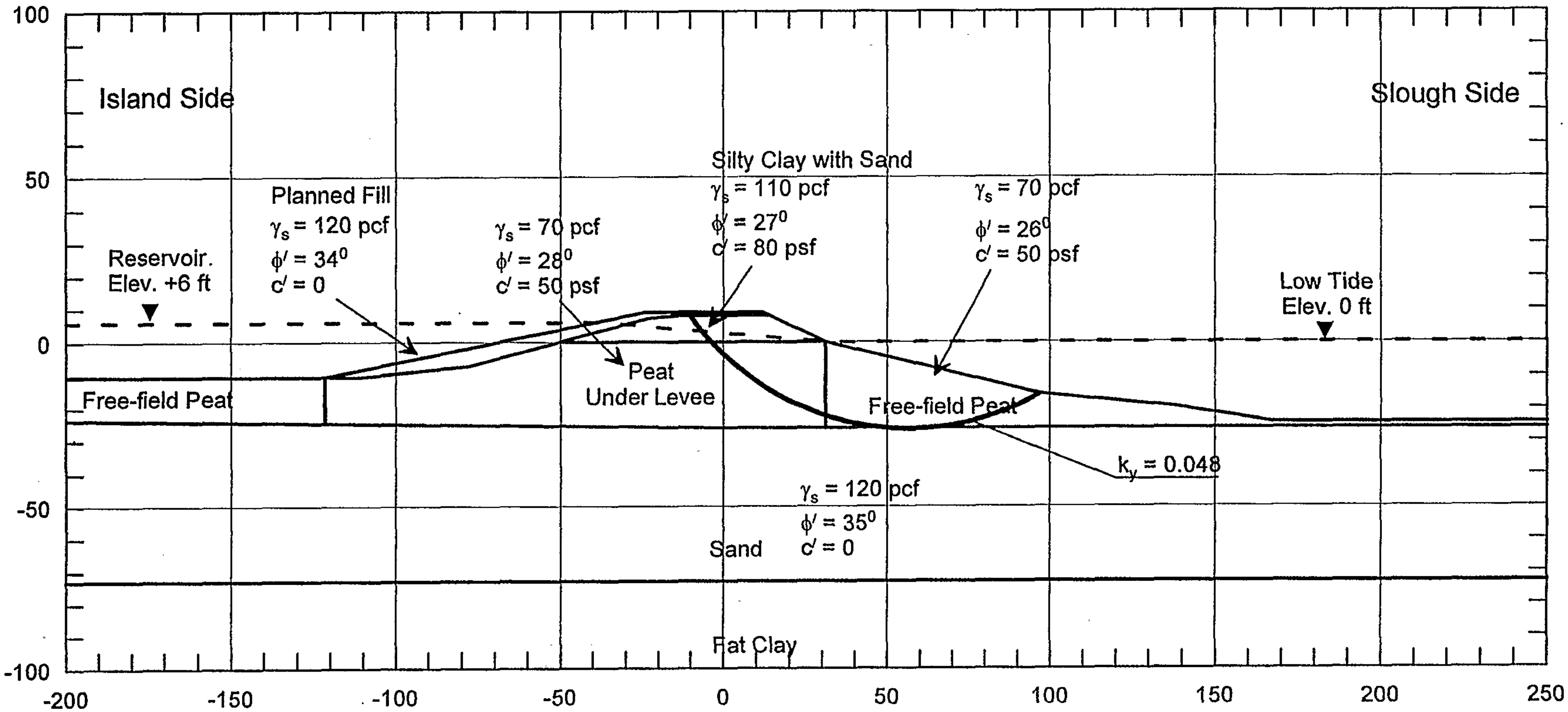
C—063504



DELTA WETLANDS PROJECT	**BACON ISLAND STA. 25+00** **STABILITY ANALYSIS** **SEISMIC CONDITION- INTO ISLAND**	**FIGURE** **3.5.34**
URS GREINER WOODWARD CLYDE		

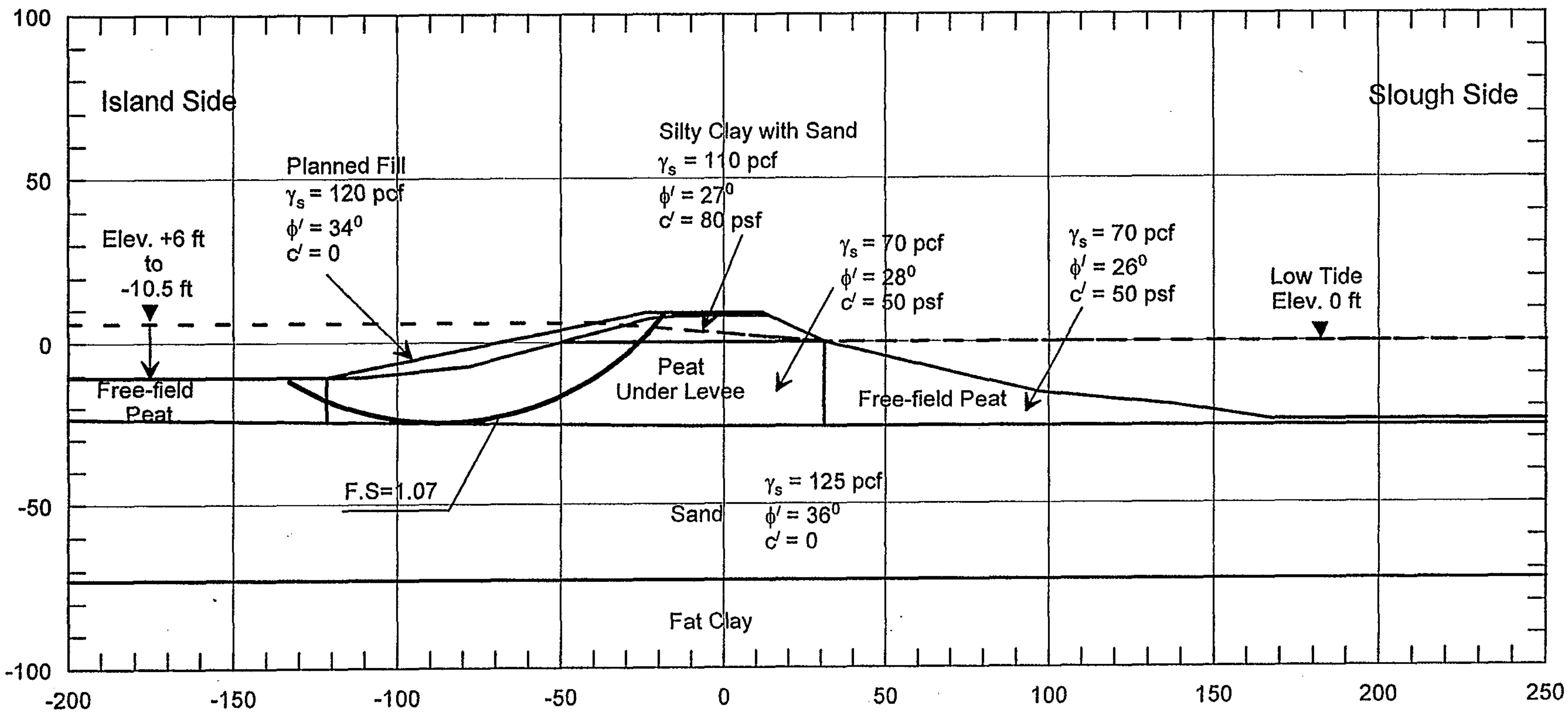
x:/ARULNA/deltawetland/Bacon25/ba25sels.grf 12/2/99

C — 0 6 3 5 0 5



DELTA WETLANDS PROJECT	BACON ISLAND STA. 25+00 STABILITY ANALYSIS SEISMIC CONDITION- TOWARD SLOUGH	FIGURE 3.5.35
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon25/ba25sesl.grf 12/2/99



DELTA WETLANDS PROJECT	**BACON ISLAND STA. 25+00 STABILITY ANALYSIS, THREE-STAGE SUDDEN DRAWDOWN CONDITION - INTO ISLAND**	**FIGURE 3.5.36**
URS GREINER WOODWARD CLYDE		

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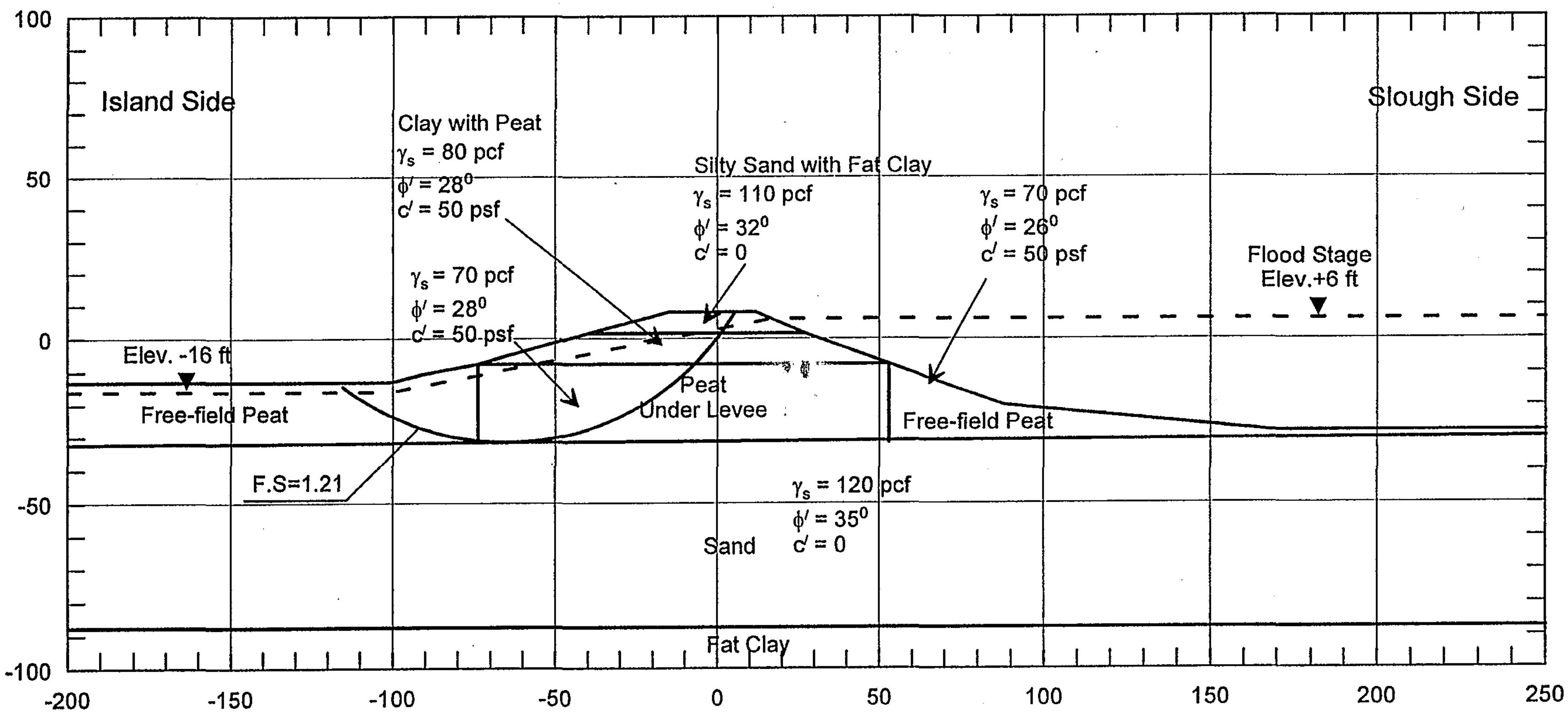
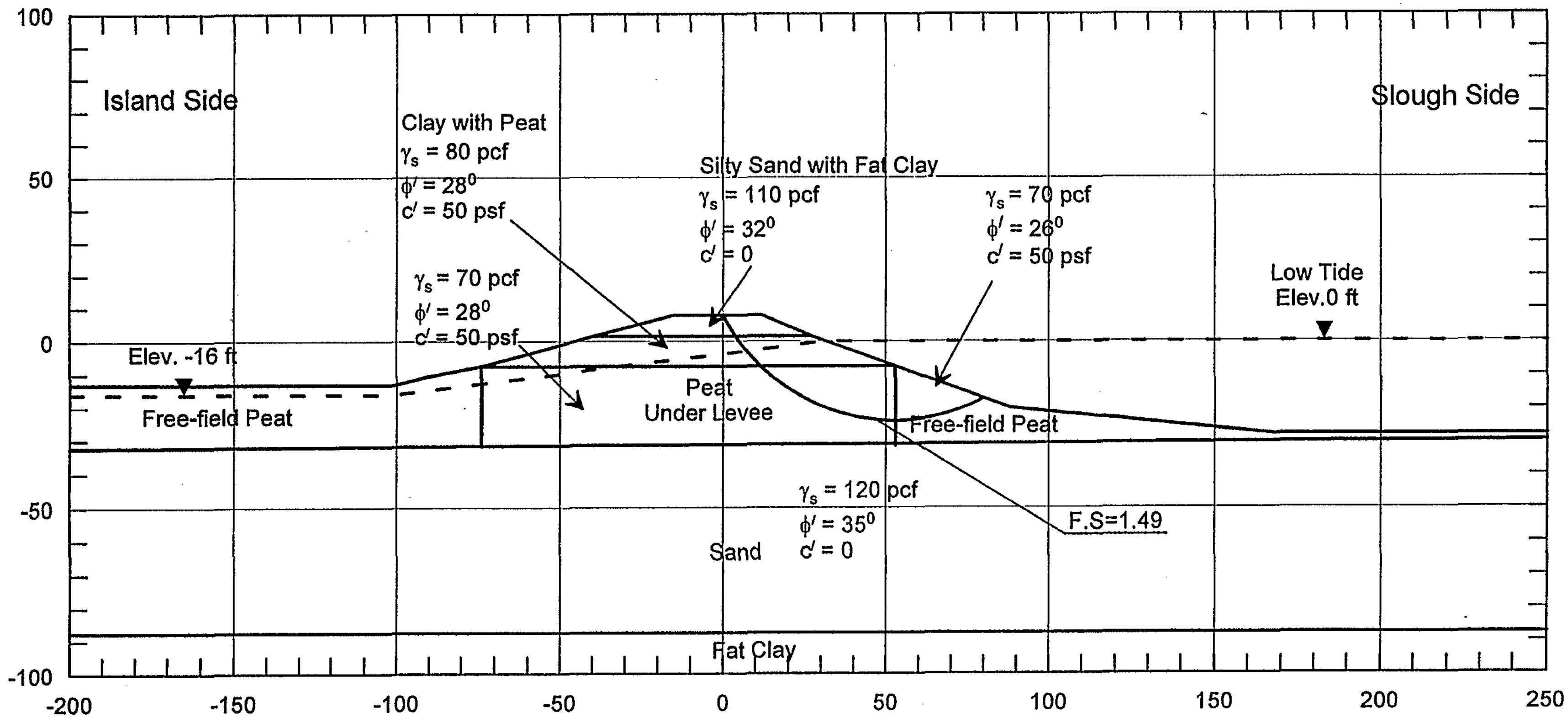


DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS EXISTING CONDITON- INTO ISLAND	FIGURE 3.5.37
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon265/ba2isl.grf 12/2/99

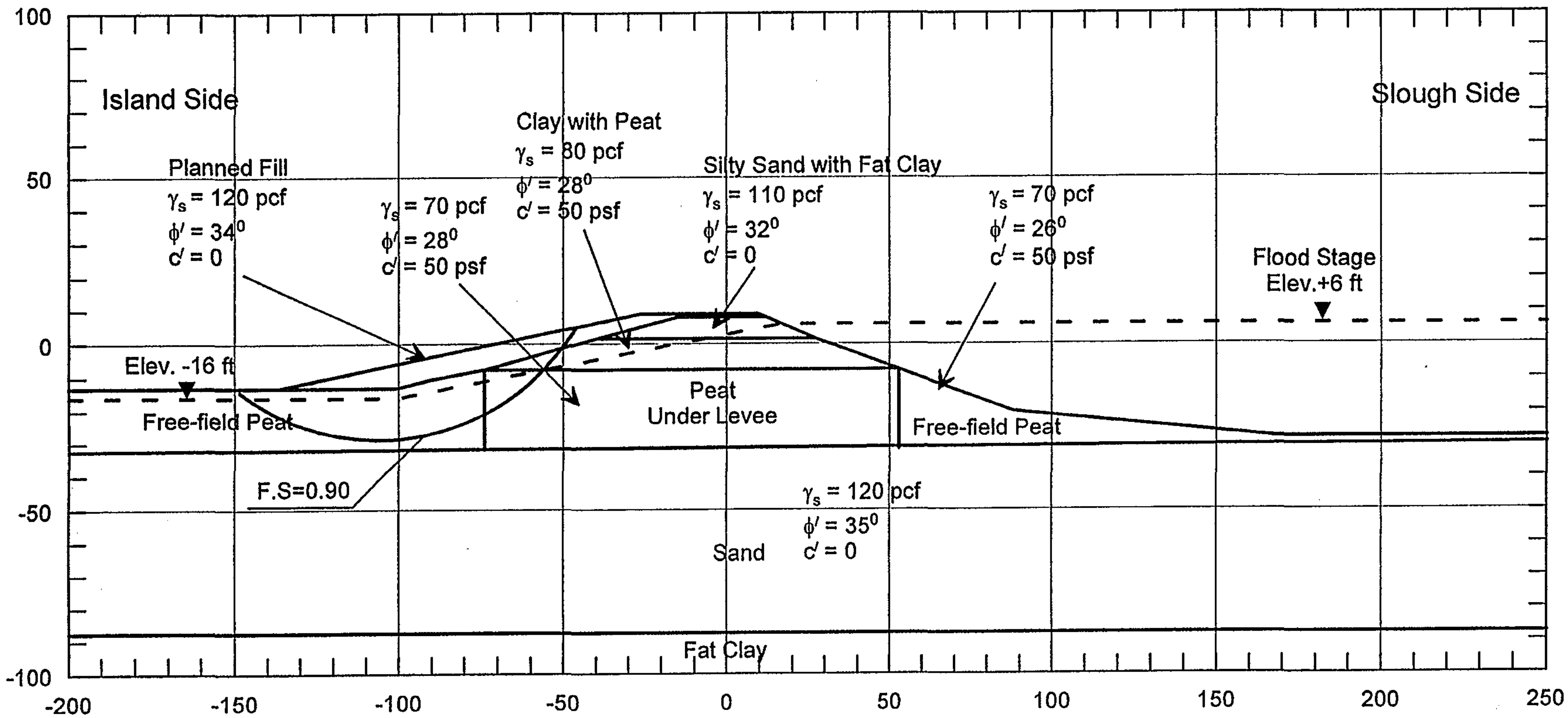


DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS EXISTING CONDITON- TOWARD SLOUGH	FIGURE 3.5.38
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon265/ba2slo.grf 12/2/99

C—063509

C-063510



DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS, TWO-STAGE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.39
URS GREINER WOODWARD CLYDE		

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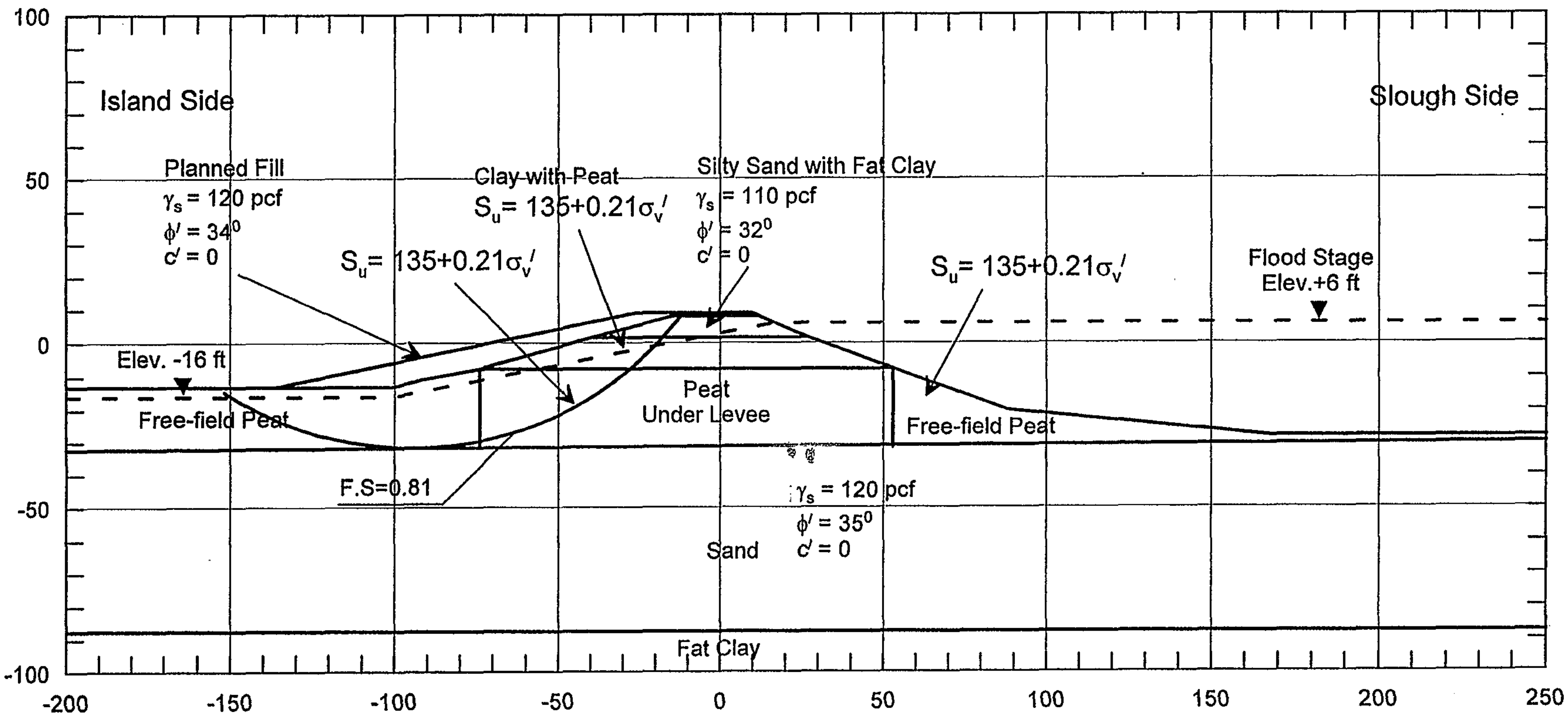
C-063510

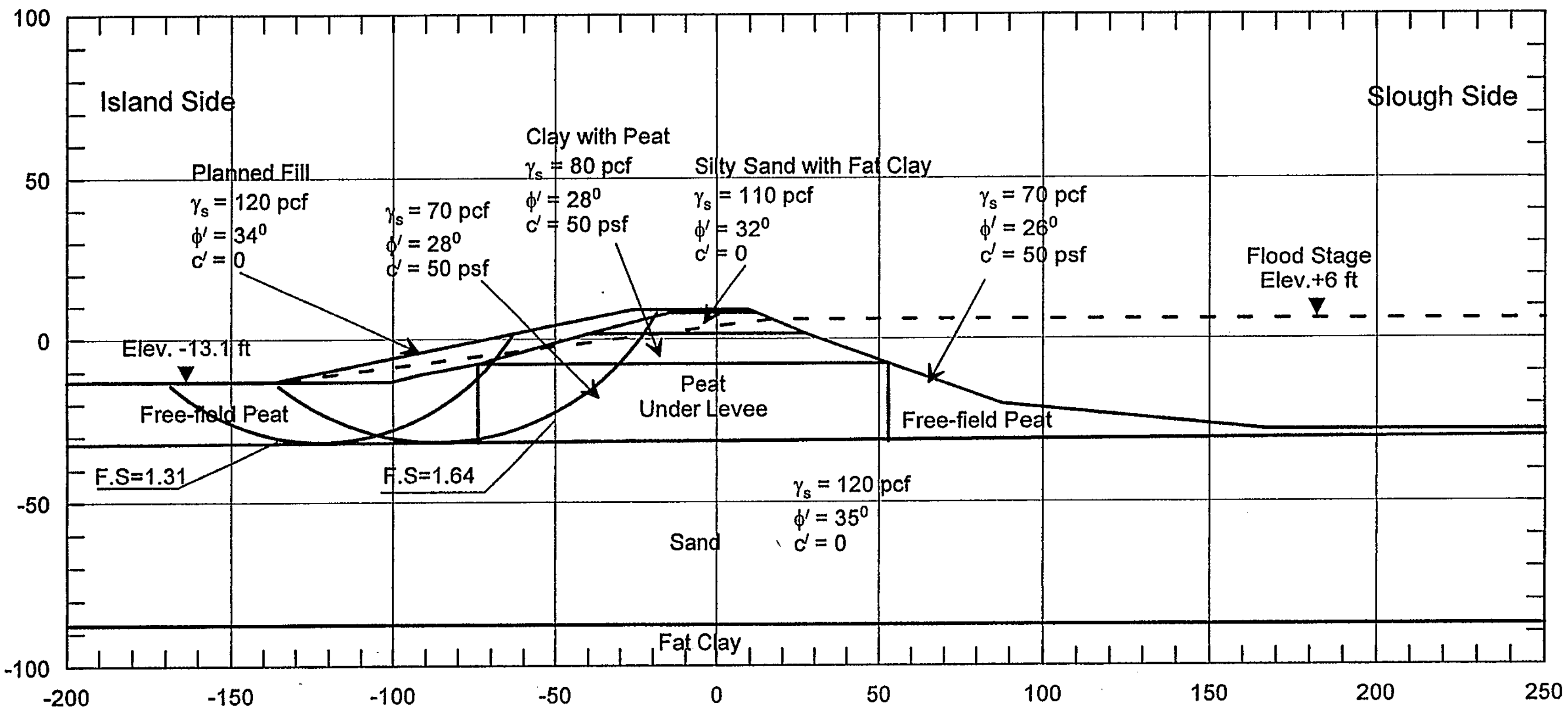


DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS, S_u PROFILE END OF CONSTRUCTION- INTO ISLAND	FIGURE 3.5.40
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon265/ba2sul.grf 12/2/99

C —0 6 3 5 1 1

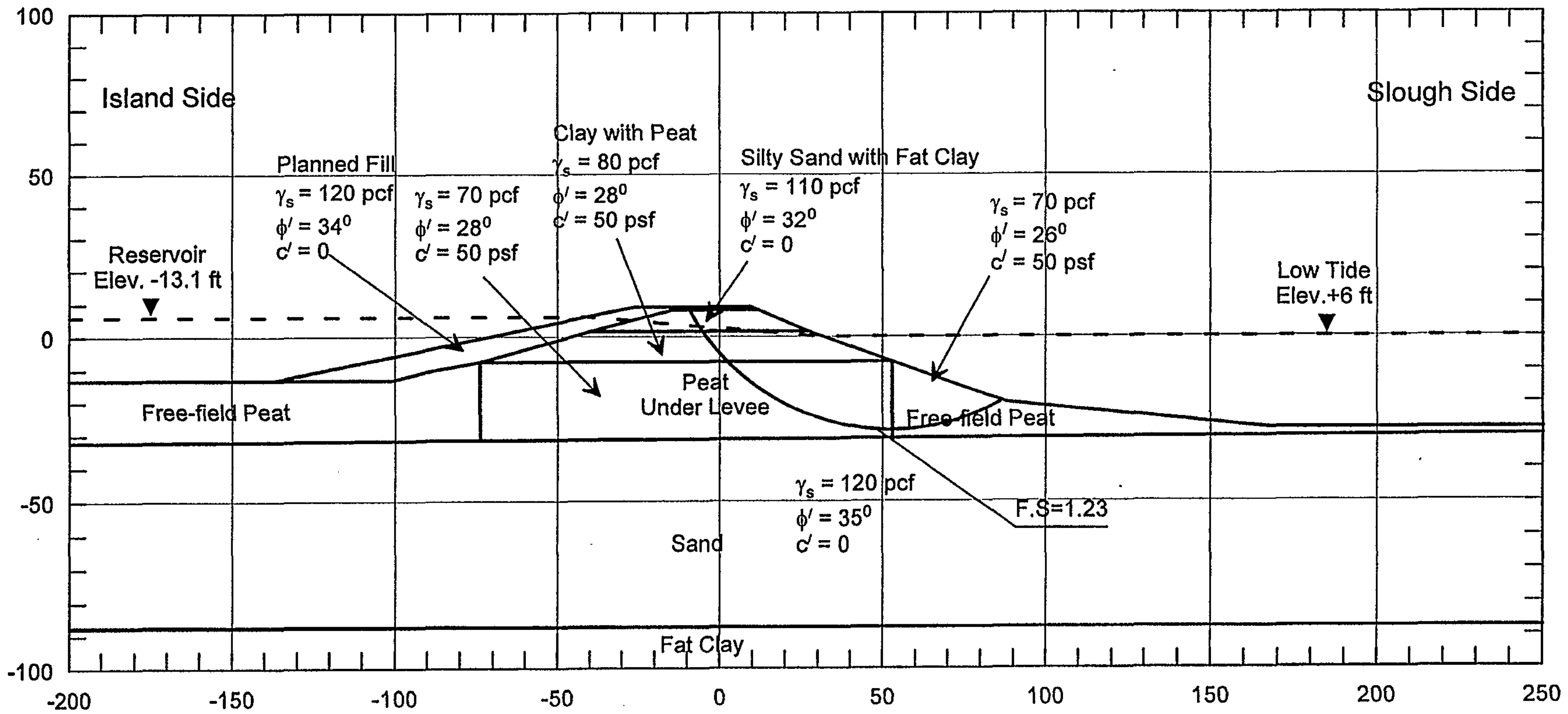
C-063511



DELTA WETLANDS PROJECT	**BACON ISLAND STA. 265+00 STABILITY ANALYSIS LONG-TERM CONDITION- INTO ISLAND**	**FIGURE 3.5.41**
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon265/ba2long.grf 12/2/99

C — 0 6 3 5 1 2



DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS LONG-TERM CONDITION- TOWARD SLOUGH	FIGURE 3.5.42
URS GREINER WOODWARD CLYDE		

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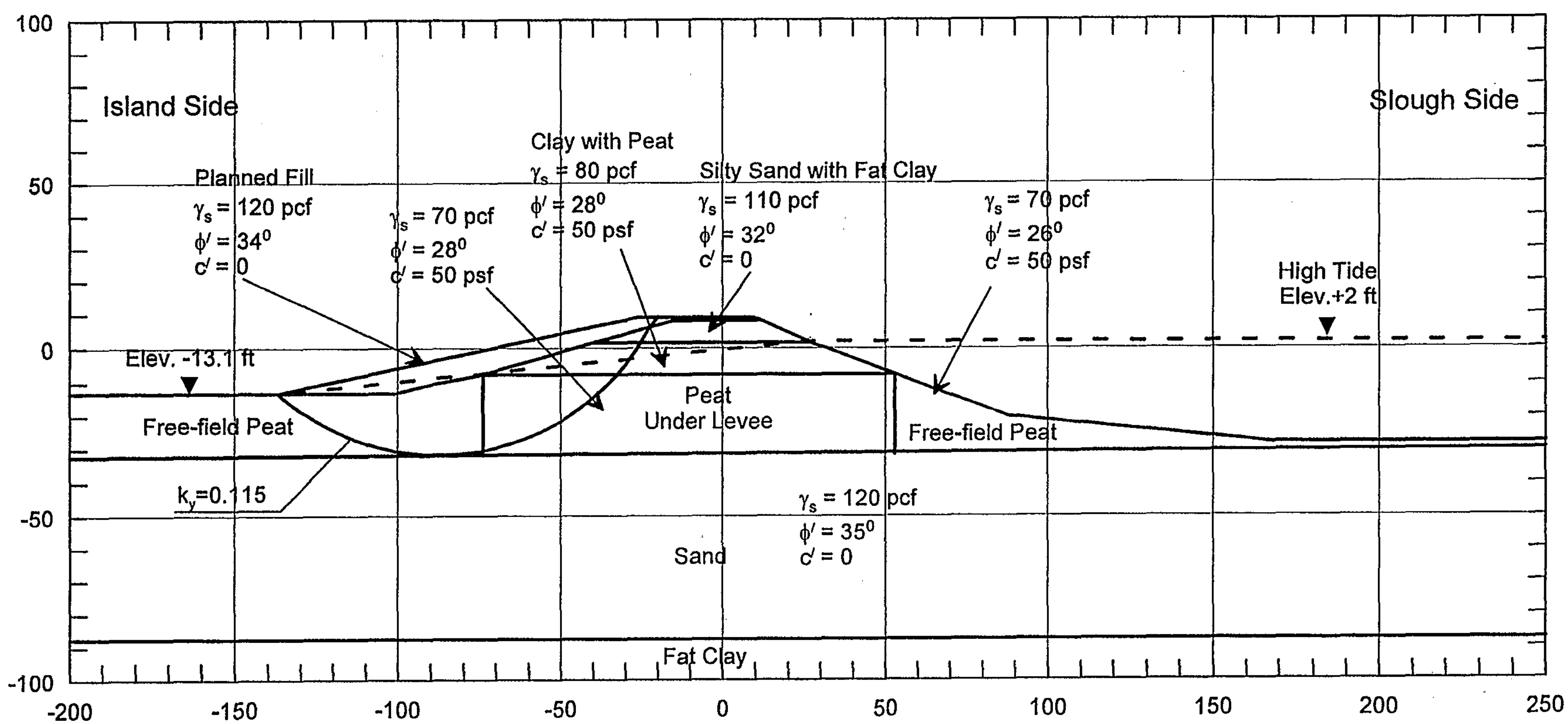
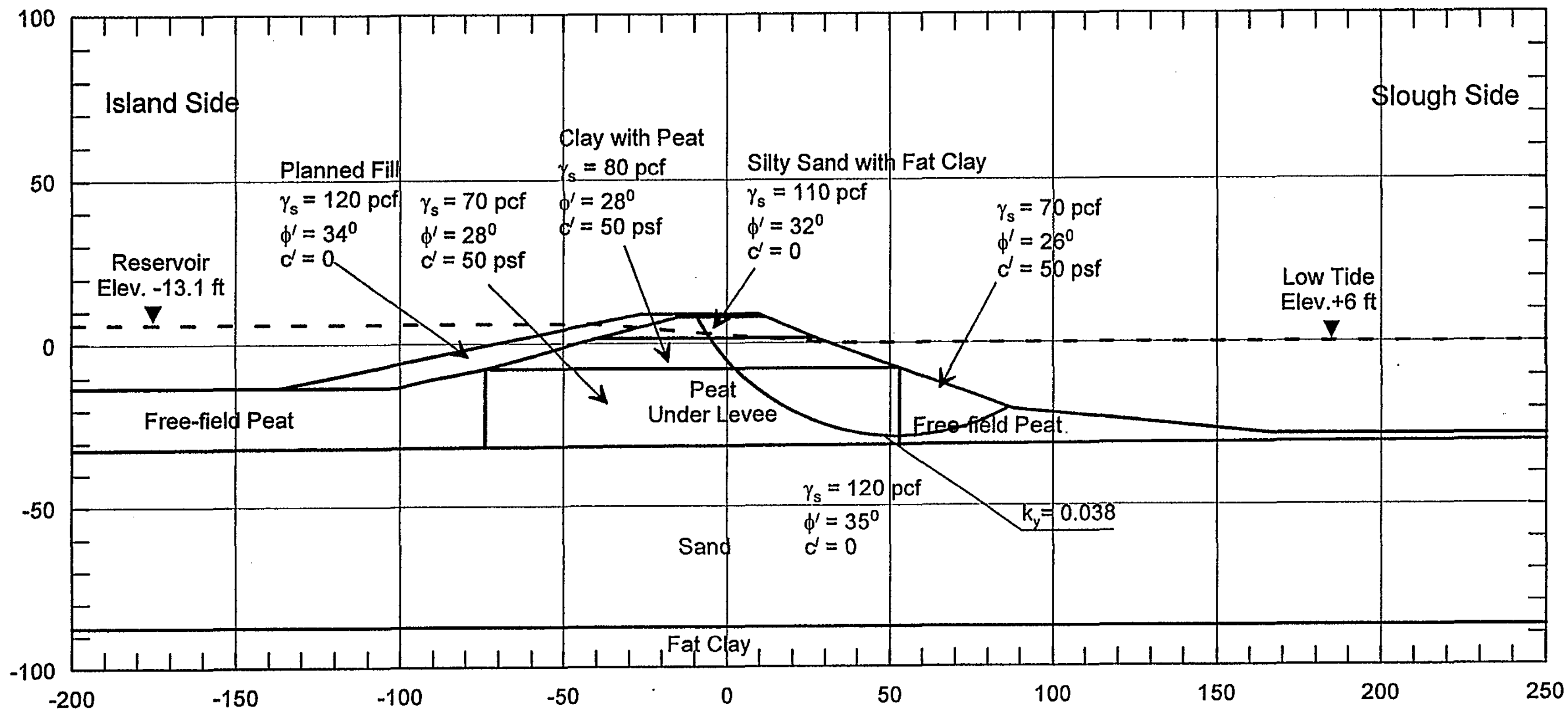


DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS SEISMIC CONDITION- INTO ISLAND	FIGURE 3.5.43
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon265/ba2sels.grf 12/2/99

C — 0 6 3 5 1 4

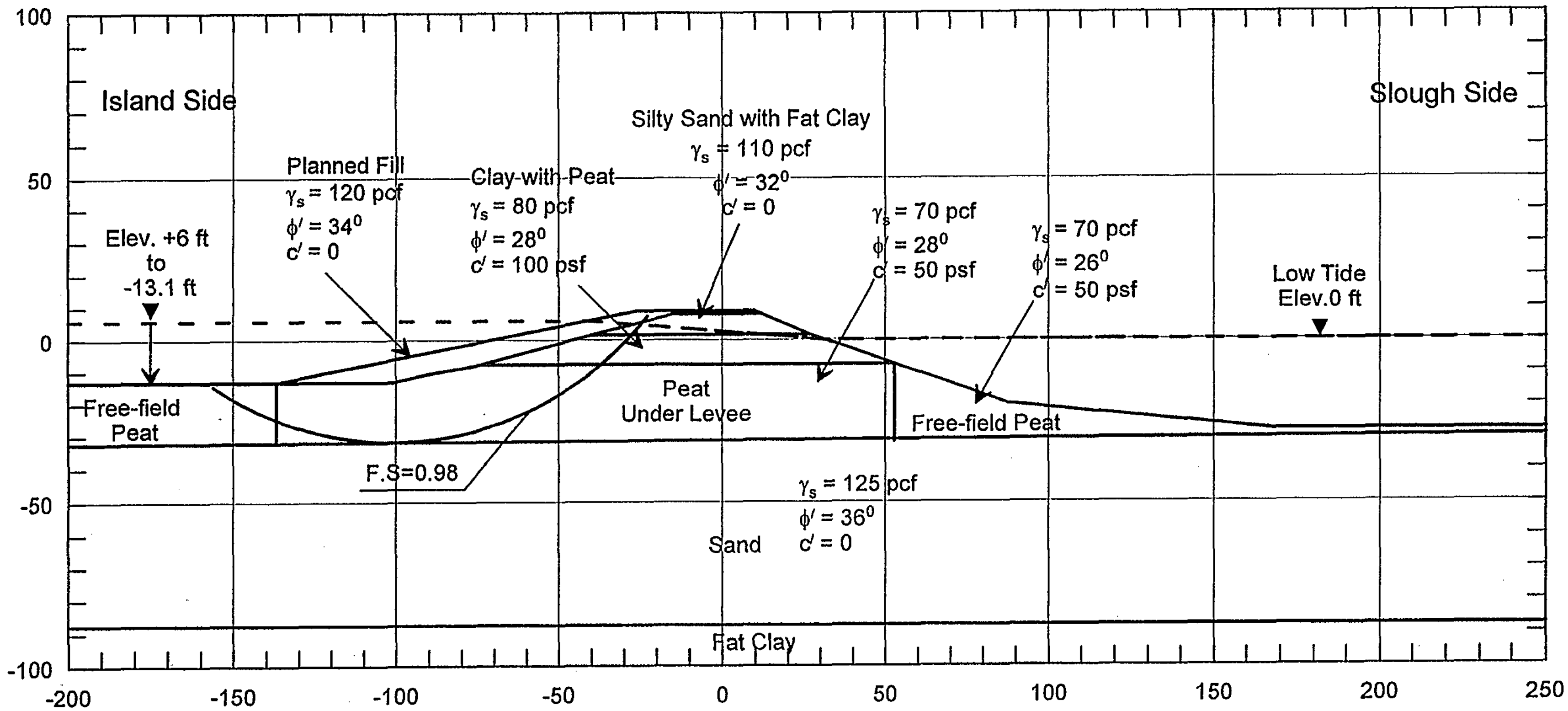


DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS LONG-TERM CONDITION- TOWARD SLOUGH	FIGURE 3.5.44
URS GREINER WOODWARD CLYDE		

x:/ARULNA/deltawetland/Bacon265/be2seat.grf 12/2/99

C — 0 6 3 5 1 5

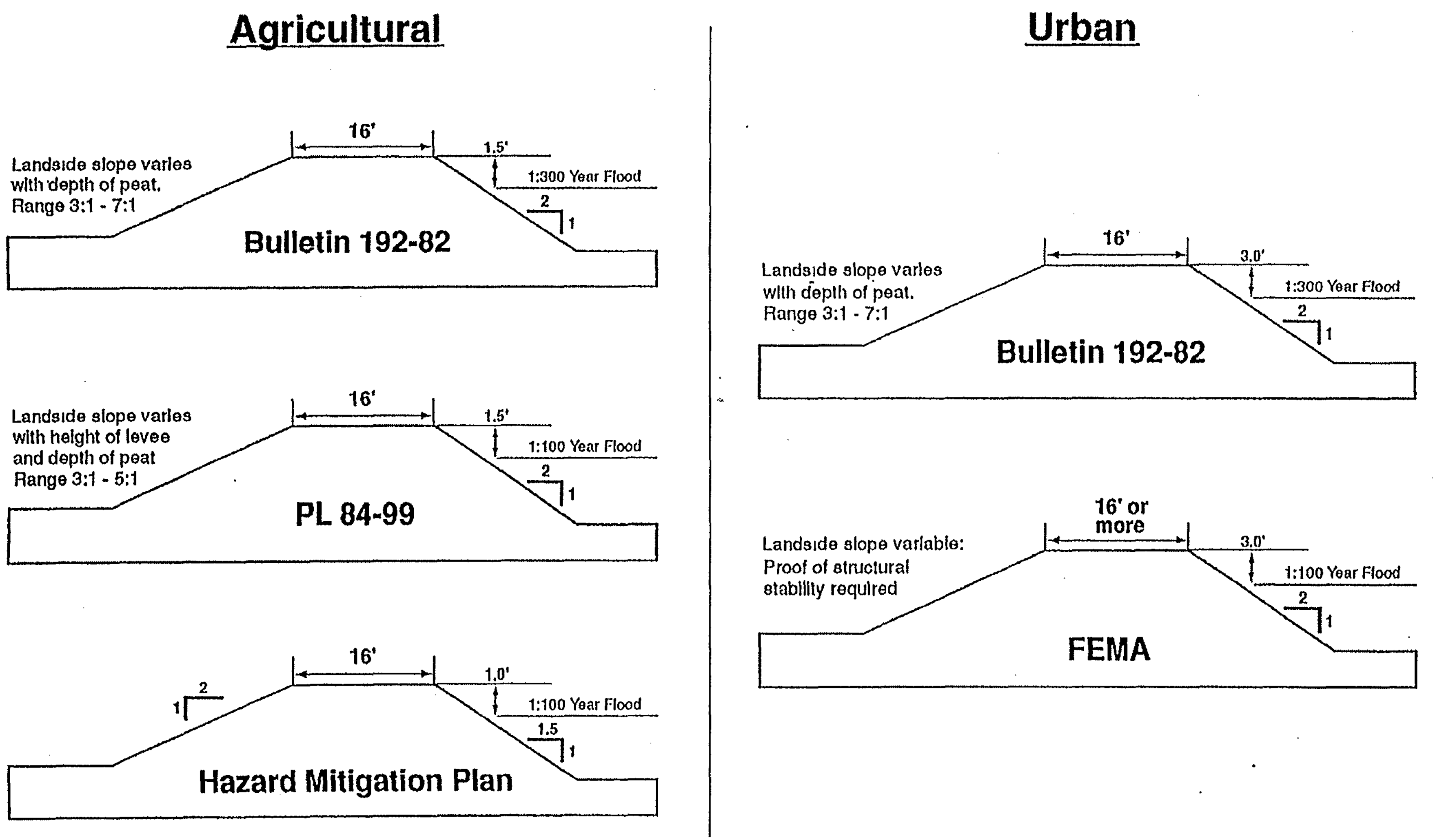
C-063516



DELTA WETLANDS PROJECT	BACON ISLAND STA. 265+00 STABILITY ANALYSIS, THREE-STAGE SUDDEN DRAWDOWN CONDITION - INTO ISLAND	FIGURE 3.5.45
URS GREINER WOODWARD CLYDE		

x:\ARULNA\deltawetland\Bacon265\basap.grf 12/2/99

C — 0 6 3 5 1 6

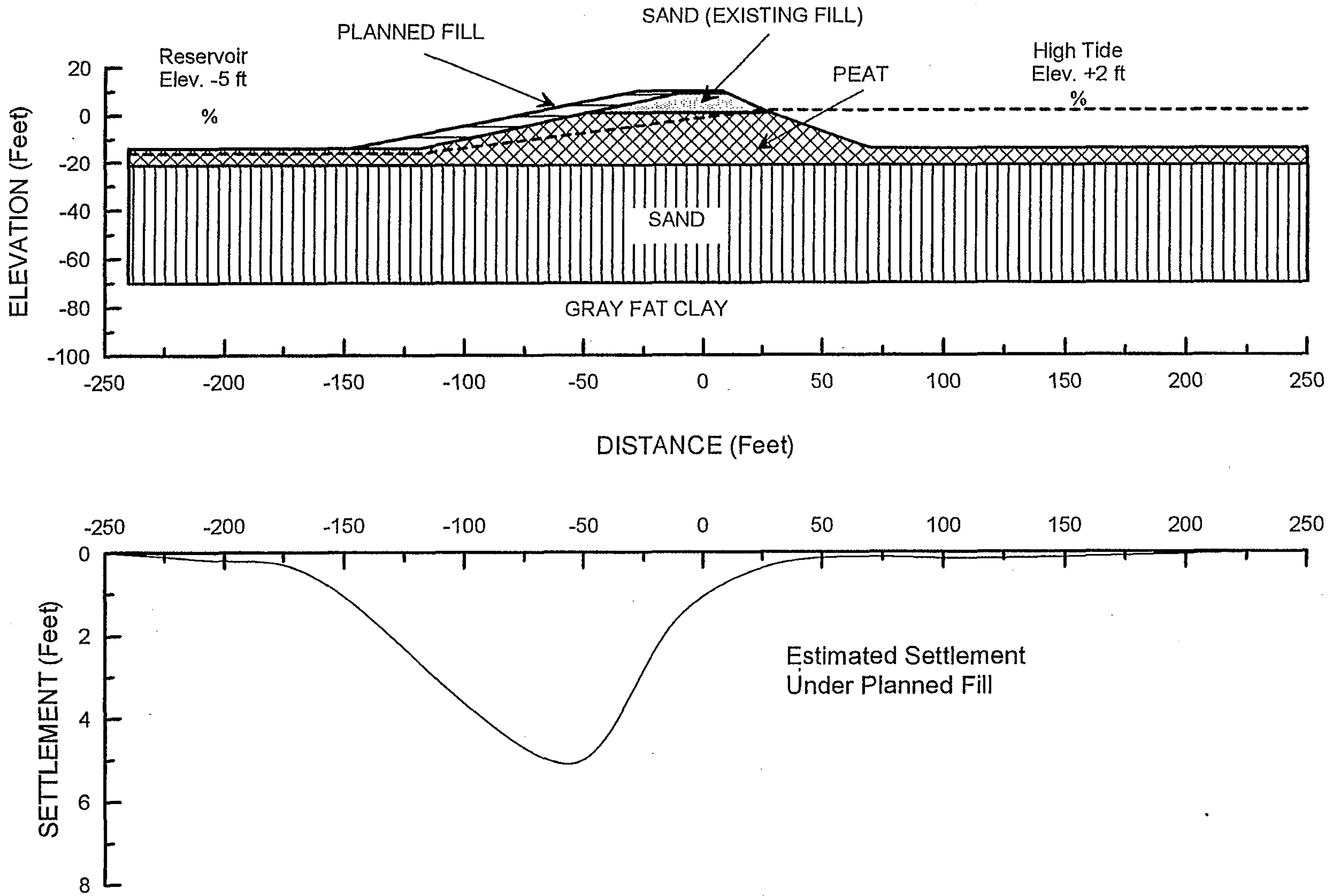


Project No. 41-07099030.00	Delta Wetlands	LEVEE GEOMETRIC STANDARDS	Figure 3.5.46
URS Greiner Woodward Clyde			

41-07099030.00-00003/120999/gos

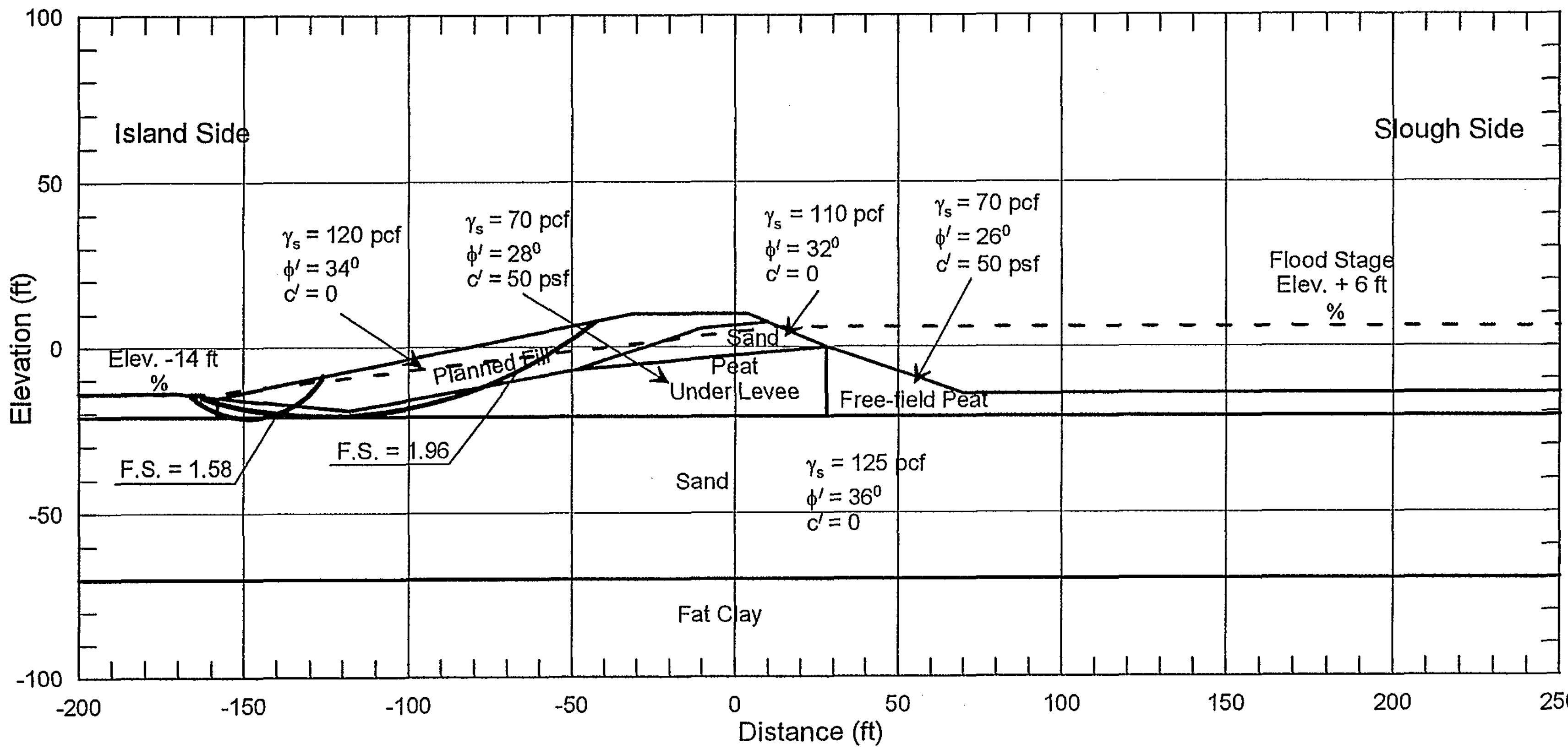
C-063517

C-063517



DELTA WETLANDS PROJECT	**WEBB TRACT STA. 630+00 SETTLEMENT ANALYSIS**	**FIGURE 3.7.1**
URS GREINER WOODWARD CLYDE		

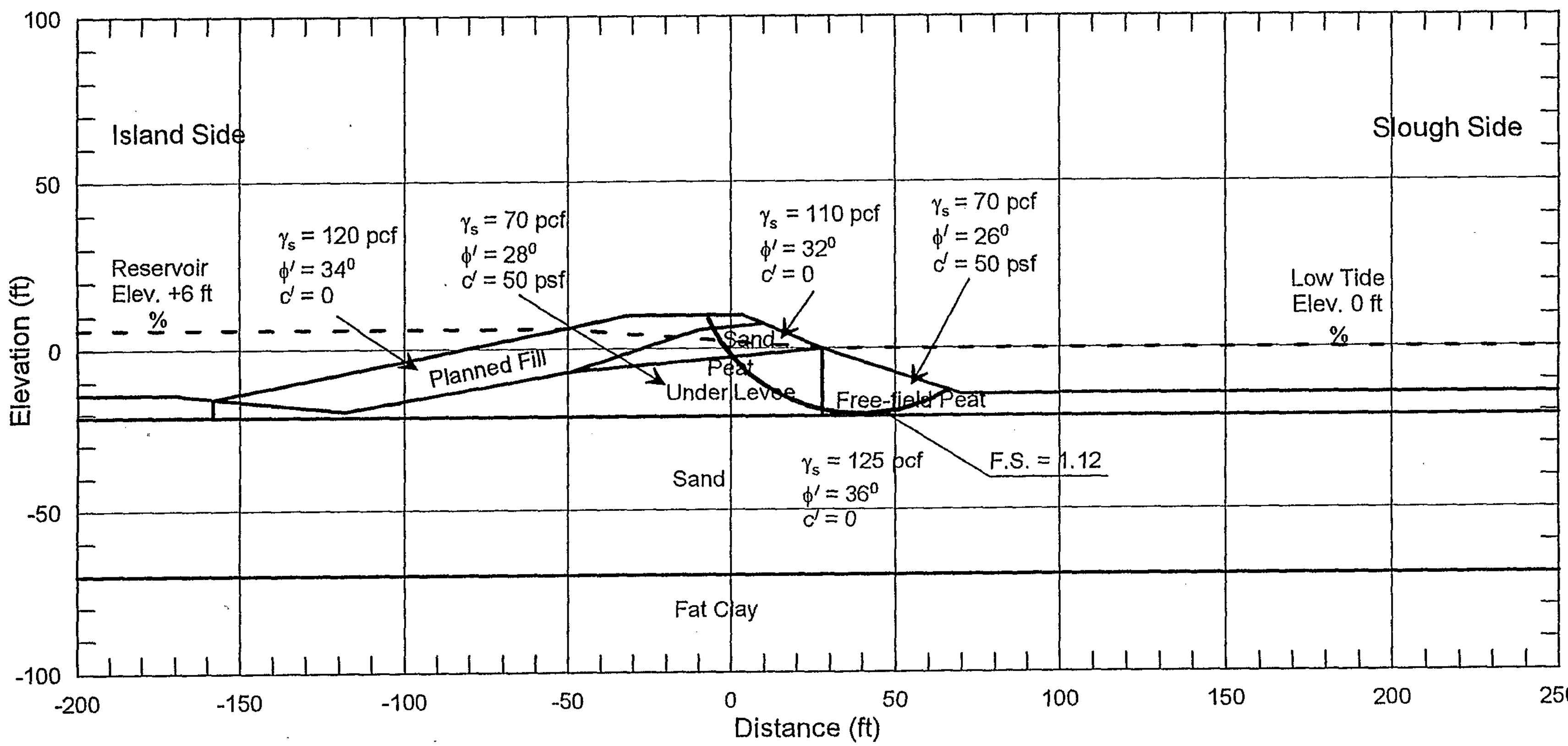
C-063519



DELTA WETLANDS PROJECT	WEBB TRACT STA. 630+00 STABILITY ANALYSIS LONG-TERM CONDITION USING DEFORMED GEOMETRY- INTO ISLAND	FIGURE 3.7.2
URS GREINER WOODWARD CLYDE		

x:/deltawetland/200files/fig372.grf 3/15/00

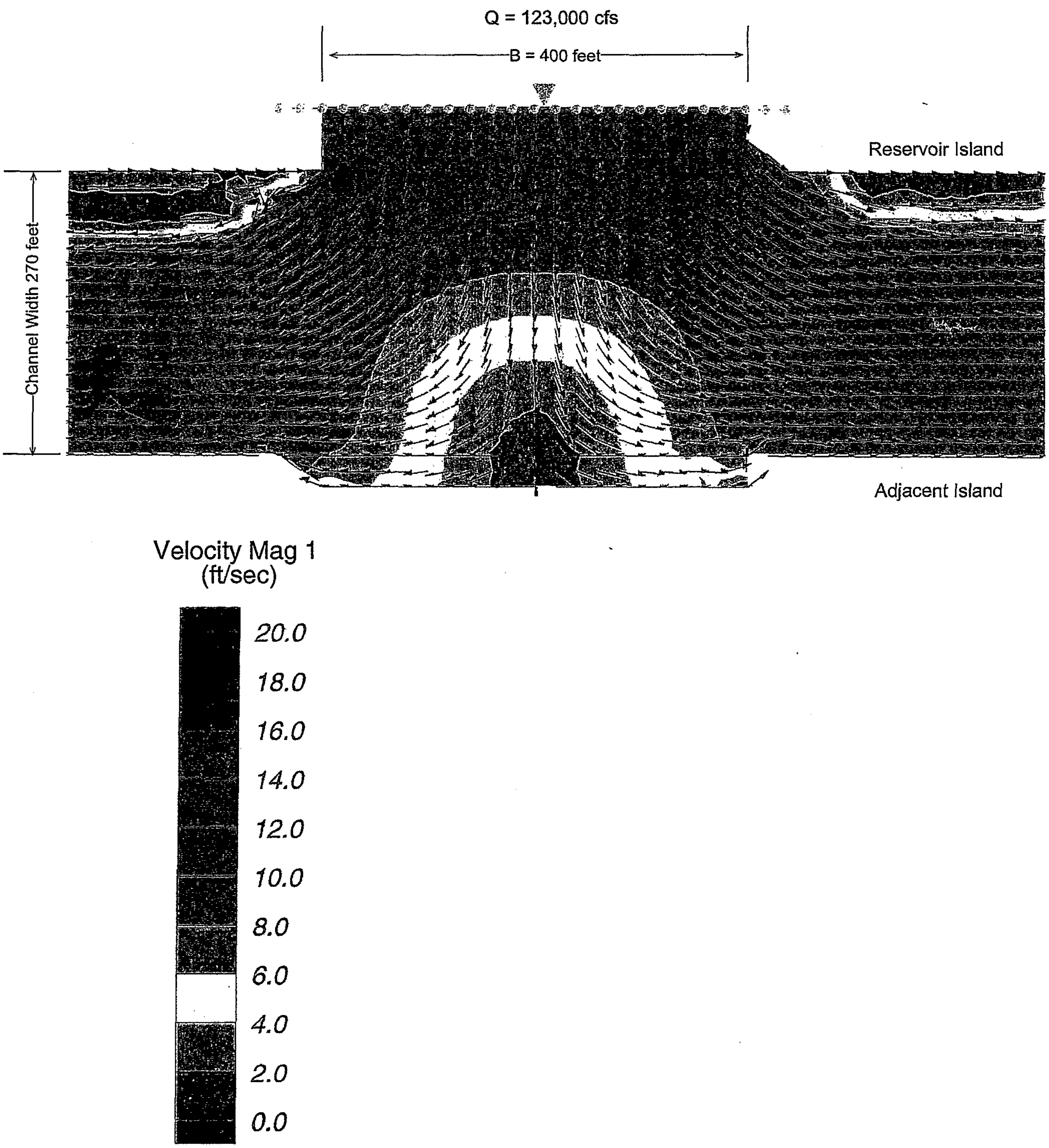
C—063519



DELTA WETLANDS PROJECT	**WEBB TRACT STA. 630+00 STABILITY ANALYSIS LONG-TERM CONDITION USING – DEFORMED GEOMETRY- TOWARD SLOUGH**	**FIGURE 3.7.3**
URS GREINER WOODWARD CLYDE		

x:/deltawetland/2000files/fig373.grf 3/15/00

C —0 6 3 5 2 0



Project No. 41-07099030.00	DELTA WETLANDS PROJECT	RESULTS OF LEVEE BREACH ANALYSIS	Figure 3.11.1
URS Greiner Woodward Clyde			

41-07099030.00-00003/051800/gos

The main findings from the seepage and stability evaluations are summarized below.

4.1 SEEPAGE ISSUES

The findings from the seepage analysis were based on two representative cross section for each island. The cross sections at each island were selected for the "narrowest" and "widest" slough width across reservoir island and neighboring island. These cross sections represent somewhat of a bounding of the seepage conditions. The following major findings emerged from the seepage evaluations.

- The proposed reservoir islands can have undesirable seepage flooding effects on adjacent islands if seepage mitigation measures are not considered.
- Seepage control by interceptor wells placed on the levees of the reservoir islands, as proposed by DW, appears effective to control undesirable seepage effects. Required well spacings and pumping rates appear to be reasonable and manageable.
- Interceptor well pumping must be carefully monitored by observation wells and application of "significance criteria."
- Use of a combination of seepage monitoring wells and background wells, as proposed by DW, combined with the use of "significance criteria," appears suitable to control the interceptor well pumping. Additional rows of shallow monitoring wells are recommended across each neighboring island. Well readings by means of automatic data acquisition are appropriate.
- Significance criteria have been developed by DW in consultation with others to apply the monitoring results to trigger seepage mitigation, consisting in the first place in pumping from the interceptor wells. The concept and format of the significance criteria appear appropriate, but some changes in the criteria appear desirable.
- The significance criteria should be re-evaluated and updated periodically.
- A system of checking the performance of individual wells and of well maintenance needs to be developed and implemented. Well maintenance should be documented and tracked, to identify wells requiring excessive maintenance.
- It appears that the most effective pumping of the interceptor wells, combined with return of some of the pumped water back into the channel (max 8% of total pump volume), will not lead to water diversion from the channel into the island.
- Operation of the reservoir islands will lead to only small additional settlements, smaller than the settlements that the islands would experience with continued use as farmland.
- Wind-induced waves during reservoir operation are expected to be significant enough to require scour and erosion protection of the inner levee slopes.
- A minimum of 800 to 1,000 feet offset form the levee toe should be maintained for the location of borrow sites. With this offset, there is no discernible effect of the borrow areas on seepage.

- The sensitivity analysis considered the channel silt permeability, aquifer permeability, and the thickness of the peat layer within the reservoir island. The results indicated that the permeabilities of the channel silt and the aquifer have a significant impact on the seepage conditions and required pumping volume, while the peat thickness has little effect.

4.2 STABILITY ISSUES

The stability of the project's levees has been evaluated by extensive stability analyses of sections selected to be representative of the more severe stability situations expected at the reservoir islands. The calculated factors of safety have been compared to various published stability criteria, and judgments were made of the adequacy of the proposed project in regard to levee stability. The resulting conclusions and recommendations are:

- The levee strengthening measures conceptually proposed by DW are generally appropriate and adequate to provide adequate stability of the reservoir islands' levees, except as noted below.
- Similarly, the seepage monitoring and control measures are generally adequate to avoid reducing the stability of adjacent islands' levees, provided the measures noted in Section 4.1. are implemented.
- Construction of the levee strengthening fills must be implemented in a manner to prevent stability failures due to the new fill loads. This will require carefully planned and timed multi-stage construction, and monitoring to observe the behaviors as the fill is placed. The staged construction will require a construction period estimated to extend over 4 to 6 years, depending on final design.
- Long-term stability toward the slough side will be reduced by the construction and reservoir filling to an excessive degree. Measures should be provided to improve this stability. Some conceptual slope stabilization measures may include:
  - Flattening the slough side levee slope
  - Widening of the levee crest to provide redundant levee width
  - Rock buttressing the levee toe on the slough side
- There may be potential environmental impact resulting from slough sideslope failure. These are outside the scope of this work and consequently are not addressed in this report.
- Stability with respect to sudden drawdown of the water in the reservoir may be inadequate at some locations. This potential failure mode should be considered carefully, and remedial measures (such as flattening of levee slopes) implemented where locally needed.
- The seismic stability evaluation of the reservoir islands levees indicates that as much as 2 feet of downslope deformation on the reservoir side and 4 feet of deformation on the slough side could be experienced during a probable earthquake in the region. The measures noted above to improve the slough side stability will also mitigate the slough-side deformation.
- As indicated by DW, it is planned, as a part of final design, to implement extensive and detailed subsurface exploration programs along the reservoir island levees, followed by stability evaluations and site-specific detailed design and construction to provide adequate

levee stability. These steps will be essential to achieve adequate safety and effectiveness of the proposed levee system.

4.3 OVERALL FINDINGS

Taking a broader view, we consider the overall findings of this reevaluation of geotechnical issues of the proposed Delta Wetlands Project to be as follows:

- The seepage mitigation design proposed by DW appears appropriate and has the potential to be effective, provided that
  - the interceptor well system is appropriately designed, constructed and operated;
  - the monitoring system consisting of seepage monitoring wells and background wells is appropriately designed, constructed and operated;
  - the significance criteria are rigorously applied and continually updated based on experience.
- The levee strengthening conceptually proposed by DW appears appropriate, except that measures need to be developed to improve the stability of the raised levee stoward the slough.
- Because conditions around the islands' perimeter vary, it will be essential that a "mile-by-mile" geotechnical exploration and, based on it, a detailed final design, be implemented. The exploration should consist of borings and soundings spaced closely enough that adverse conditions extending over some distance would be identified. Appropriate detailed geotechnical laboratory tests, in particular grain size, permeability and strength tests, should be made on recovered samples. Final design of seepage control and monitoring, and levee strengthening, should consider the specific conditions identified on a site-specific basis.
- Construction of the improvements will require detailed geotechnical construction oversight, construction quality control and quality assurance, and documentation of as-built features, to maximize the chances that unexpected conditions are identified and accommodated, that construction will be implemented to satisfy the intent of the design, and that construction is documented.
- In particular, extraction well design, construction and operation will be critical not only to maximize the reliability of the seepage control system, but also to minimize the possibility of flushing fine particles out of the levee foundation, which could over time lead to weakened levee foundations and potential settlement and stability problems. Experience has shown that this can be achieved.
- It is recognized that pumping from the crest of the reservoir levee to mitigate seepage effects across the slough in the adjacent island is not the most effective way to achieve the seepage mitigation. It has been selected because of ownership and access issues. Other measures to achieve the seepage mitigation could be developed. In particular, pumping from the adjacent island's levee across the slough from the reservoir island would be hydraulically more efficient, and would likely require fewer wells and lower pumping volume. Passive or active relief wells or trenches on the adjacent island would also be effective. A continuous cutoff around the reservoir islands would also be effective, but would likely be cost prohibitive.

The supplemental geotechnical evaluations described in this report had a number of limitations. We made only reconnaissance-type visits to the two islands considered for reservoir islands that we evaluated. No additional site and subsurface investigations were made. Consequently, our work was based on existing, widely spaced borings and cone penetration tests; and on available laboratory tests made by Harding Lawson Associates. Most of the laboratory tests were also made more than 10 years ago and may not have used most current testing procedures. Thus, we had to rely for levee geometries, levee and subsurface soil conditions, and soil seepage and strength properties on information developed by others. We also relied on survey data by others.

Our evaluations were made on two cross sections on each proposed reservoir island. These cross sections, which were mostly different for seepage and stability evaluations, were selected based on available data to be reasonably representative but on the conservative side for seepage and stability issues, respectively. The most severe conditions that may be encountered may not have been considered. Nevertheless, the results for the sections that were analyzed suggested in all cases that more severe conditions could be accommodated with suitable changes in the design. Such accommodation will need to be considered in the final design.

Finally, this project was implemented with the care normally associated with work of this kind in this area at the present time. No other warranty is given or implied.

An Annotated List of Geotechnical Reports Prepared for the DW Project by Harding Lawson Associates between February 1989 and November 1993 is contained in Appendix D1 to the Draft EIR/EIS (Jones and Stokes, 1995). This annotated list is attached to this report at the end of Section 6.0. The references contained in the annotated list are not repeated in the Section 6.0 listing. They are referred to in the text of the report by the standard method (e.g., HLA 1992a). All other references are listed below.

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Appendix D1. Annotated List of Geotechnical Reports Prepared for the DW Project

Harding Lawson Associates, Inc. 1989. Preliminary geotechnical investigation for the Delta Wetlands project. By K. Tillis, E. Hultgren, and C. Wood. February 15, 1989. (HLA No. 18749,001.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents the results of a preliminary geotechnical investigation performed by Harding Lawson Associates (HLA) for the Delta Wetlands (DW) project. The investigation was to provide preliminary geotechnical design for the project. The report describes the results of collecting available data on soil conditions and physical properties of Delta levees and foundation materials and of exploring subsurface conditions to define site stratigraphy and obtain soil samples for visual observation and laboratory testing. The report also provides preliminary conclusions and recommendations regarding geotechnical engineering concerns.

HLA's field investigations consisted of drilling, logging, and sampling exploratory borings; performing cone penetration test soundings; and installing and monitoring piezometers at representative locations around the island perimeters. Soil samples were collected and analyzed from levees and levee foundations on each of the project islands. Soil tests included particle size analyses, consolidation tests, and the determination of soil moisture content, dry density, shear strength, and permeability. The effects of levee reconstruction on levee settlement were estimated from the boring data, soil sample consolidation test results, and published data on settlement of fill material placed on peat soils of the Delta. HLA analyzed slope stability toward island interiors and toward Delta channels for the existing, after-construction, and long-term conditions.

__________. 1990a. Project status report: McDonald Island drawdown demonstration. By K. Tillis, D. Holloway, and E. Hultgren. February 22, 1990. (HLA No. 18749,013.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report summarizes results of the McDonald Island drawdown demonstration study. The purpose of the investigation was to demonstrate that hydraulic head within the sand aquifer can be lowered by pumping through a groundwater relief well system, and that similar systems would be viable options for controlling seepage resulting from the operation of DW reservoirs. HLA conducted a field investigation to confirm stratigraphy and install observation piezometers and the relief well system between July 10 and September 1, 1989. Water levels were then monitored before, during, and after the pumping phase of the demonstration (November 14, 1989, to January 24, 1990). The report concludes that pumping is effective in controlling essentially all seepage into the island, as indicated by the flattening of the hydraulic grade line beneath the island interiors.

__________. 1990b. Groundwater data transmittal, Delta Wetlands monitoring program. By D. Holloway, K. Tillis, and E. Hultgren. April 12, 1990. (HLA No. 18749,007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents groundwater monitoring data collected through March 1990 for a groundwater monitoring program performed by HLA for the DW project. The groundwater monitoring program is to provide baseline information on existing groundwater levels in the Delta. Data were obtained from a network of piezometers installed to monitor pore pressure (i.e., hydraulic head) within the sand aquifer at varying locations on the DW islands and other Delta islands. Water levels were measured weekly during spring 1989, and from fall 1989 through March 1990. To supplement manual measurements, water-level data were continuously recorded for 1-2 weeks at a time. The report presents boring logs, results of grain size analyses, well completion diagrams for 27 piezometers, and data on groundwater level.

__________. 1990c. Project status report: McDonald drawdown demonstration Phase II. By K. Tillis, D. Holloway, and E. Hultgren. November 19, 1990.

(HLA No. 18749,013.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents results of a Phase II drawdown demonstration study performed by HLA for the DW project. The purpose of the Phase II study was to demonstrate that artesian head in the sand aquifer can be lowered by a groundwater gravity dewatering system for seepage control. Between June and mid-July 1990, the existing relief well system (pump system) was converted to a gravity-flow system, in which groundwater flows from wells into seepage ditches by artesian pressure in the sand aquifer. The report concludes that, based on groundwater level monitoring, the gravity flow system shows results that are similar to those of the pumped well system.

_________. 1991a. Groundwater data transmittal No. 2, Delta Wetlands monitoring program. By D. Holloway, K. Tillis, and E. Hultgren. January 7, 1991. (HLA No. 18749,007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents the status of the groundwater monitoring program described above (HLA 1990b). This report presents data collected from March to December 1990. Seven additional piezometers were installed in September 1990, resulting in a total of 34 piezometers on 17 Delta islands. Groundwater data for the piezometers from March through December 1990 are presented in this report.

_________. 1991b. Interceptor well modeling for the Delta Wetlands project. By D. Holloway, K. Tillis, and E. Hultgren. (HLA No. 18749,016.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents the results of HLA's groundwater modeling effort for the DW project. The model simulated various pumping well systems located on DW island levees for controlling groundwater flow off the island. The purpose of the study was to establish parametric relationships that could serve as the basis for conceptual design of pumping and interceptor well systems on DW islands. The goal of the modeling was to simulate groundwater withdrawal required to offset the increase in head in the sand aquifer, keeping groundwater levels on neighboring islands unaffected by water storage on the DW islands. The report describes the modeling approach and procedures and results of three conceptual aquifer system models. Results of the study provide a range of well spacing distances for corresponding ranges of aquifer properties, system dimensions, and pumping rates.

_________. 1991c. Groundwater monitoring plan for the Delta Wetlands project. By D. Holloway, K. Tillis, and E. Hultgren. January 23, 1991. (HLA No. 18749,007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This document presents a seepage monitoring plan for the DW project. The report describes the rationale for spacing of piezometers on neighboring islands. The proposed piezometer locations are shown on a regional map. Piezometers are planned for all levee reaches located across from DW reservoir islands. Additional piezometers are proposed at locations remote from the reservoirs to provide data on general Delta-wide groundwater level variations for comparison with water level fluctuations near DW reservoirs during project operation. The report describes methods for evaluating the groundwater level and outlines criteria for determining whether a net seepage impact is occurring.

_________. 1991d. Seepage control program for the Delta Wetlands project. By D. Holloway, K. Tillis, and E. Hultgren. January 24, 1991. (HLA No. 18749,007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report summarizes existing conditions on and adjacent to the DW project islands and outlines a seepage control program for the DW project. The program is based on information and recommendations presented in HLA's preliminary geotechnical investigation, McDonald Island drawdown demonstration project status reports, groundwater data transmittals, and interceptor well modeling reports. The report describes potential seepage effects of farming, wetland management, and reservoir management and outlines potential measures to control seepage, including cutoff walls, interceptor wells, and relief wells.

_________. 1992a. Wave erosion monitoring and mitigation for the Delta Wetlands project. By K. Tillis and E. Hultgren. January 6, 1992. (HLA No. 18749,007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report describes measurable performance standards, monitoring, and mitigation measures for wave erosion on the interior slopes of the DW project levees. This report assumes a spending beach design for the interior levees. (The current project description for this environmental impact report/environmental impact statement [EIR/EIS] does not include spending beach design.)

__________. 1992b. Monitoring and mitigation of geotechnical impacts on State Route 12 for the Delta Wetlands project. By K. Tillis and E. Hultgren. January 7, 1992. (HLA No. 18749, 007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents a proposed design for a new dam to impound a reservoir south of State Route 12 on Bouldin Island. The report describes proposed drainage structures, performance standards for settlement and shallow groundwater, potential and anticipated geotechnical effects of the new dam, and monitoring needs. This proposal is for the four-island, maximum fill alternative (Alternative 3).

__________. 1992c. Seepage monitoring and mitigation for the Delta Wetlands project. By K. Tillis and E. Hultgren. January 8, 1992. (HLA No. 18749,007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report provides an overview of seepage issues that affect Delta islands and how water storage on one island may affect an adjacent island. This report proposes a seepage monitoring plan and measures to mitigate seepage.

__________. 1992d. Phreatic surface in perimeter levees for the Delta Wetland project. Letter report by K. Tillis and E. Hultgren to J. Winther, President, Delta Wetlands. January 9, 1992. (HLA No. 18749,007.03.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This letter report addresses the anticipated level of phreatic surface within the buttressed perimeter levees on the DW project islands. The phreatic surface (free water surface) is the level below which groundwater would seep into an excavation, boring, or well. To estimate the phreatic surface, HLA created flow nets to assess seepage through the levee. The report describes factors affecting the phreatic surface and results of analyses conducted on Holland Tract. The report concludes that the phreatic surface would rise as fill is placed for levee reconstruction.

__________. 1992e. Geotechnical investigation and design for the Wilkerson Dam on Bouldin Island. By K. Tillis, S. Vahdani, and K. Bergman. May 27, 1992. (HLA No. 11472-008.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents the results of a geotechnical investigation and design studies for Wilkerson Dam on Bouldin Island. The purpose of the investigation was to develop design criteria appropriate for a dam that falls under the jurisdiction of the State of California (California Department of Water Resources' Division of Safety of Dams). The report describes site conditions, design considerations, and several analyses performed to design Wilkerson Dam. Two alignments were investigated in detail as part of the study.

The study included an extensive field investigation using cone penetration test probes, borings, piezometers, down-hole seismic techniques, and a test fill constructed on peat. Laboratory tests were also conducted to evaluate strength and compressibility characteristics of soft marsh deposits, grain size distribution of sandy soils, permeability of planned fill and in situ soils, and basic index properties. Results of these analyses were used to develop engineering parameters for design. This proposal is for the four-island, maximum fill alternative (Alternative 3).

__________. 1992f. Groundwater data transmittal No. 3 Delta Wetlands monitoring program. By K. Tillis and E. Hultgren. June 25, 1992. (HLA No. 18749,007.03 [11471.007].) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report presents the status of the groundwater monitoring program described above (HLA 1990b). This report presents data collected from December 1990 to October 1991. Groundwater data for the 34 piezometers discussed above are presented in this report.

__________. 1993a. Geotechnical evaluation of perimeter levees for the Delta Wetlands project. Letter report by K. Tillis and E. Hultgren to J. Winther, President, Delta Wetlands. November 16, 1993. (HLA No. 11471,007.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This letter report discusses the results of the geotechnical evaluation for perimeter levee improvements planned in response to revisions to the DW project and alternatives description. The impact of planned levee improvements on slope stability were evaluated for two

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different existing levee conditions. Changes in the factor of safety from existing conditions were computed for the revised levee reconstruction design.

_________. 1993b. Description of Wilkerson Dam on Bouldin Island for the Delta Wetlands project. Letter report by K. Tillis and E. Hultgren to J. Winther, President, Delta Wetlands. November 17, 1993. (HLA No. 11471,007.) Concord, CA. Prepared for Delta Wetlands, Lafayette, CA.

This report describes in conceptual terms the size and nature of Wilkerson Dam under the revised four-island, maximum storage alternative (Alternative 3). This information is presented in Appendix E1, "Design and Construction of Wilkerson Dam South of SR 12 on Bouldin Island".

Appendix A

Seismic-Induced Levee Deformations and Geologic Hazards

A.1 OBJECTIVE

This section presents the results of the seismic evaluation of the levees for the proposed reservoir islands of the Delta Wetlands project. The reservoir islands analyzed for this study are the Webb Tract and Bacon Island. The seismic evaluation analysis was performed for the levees' final configuration, which includes strengthening of the levee slope. New buttresses will be constructed on the reservoir-side levee slopes of the islands to increase the safety margin of the levees.

The analyses included dynamic finite element analysis and seismic geologic hazard assessment to evaluate the seismic performances and deformations of the levees under design earthquake ground motions. The analyses presented in this section were performed in accordance with the scope of services presented in Task 3, Levee Stability, of our proposal to Jones & Stokes Associates, Inc, dated July 06, 1999.

The objectives of these analyses were to: 1) review previous studies on dynamic soil properties, earthquake ground motions and dynamic levee responses; 2) review and use existing seismic hazard studies in the region and corresponding ground motions; 3) evaluate levee dynamic responses to the design earthquake motions; 4) estimate the potential deformations induced by the design earthquake; and 5) assess the potential of seismic geologic hazards.

A.2 REVIEW OF PREVIOUS STUDIES

Previous studies of the seismic hazards for the Delta Wetlands project were performed by Harding Lawson Associates (HLA, 1992e including Appendices). These studies included, among others data, the development of design earthquake ground motions, characterization of dynamic soil properties, and analysis of levee dynamic responses (site response analysis). The studies were performed for the proposed Wilkerson Dam on Bouldin Island, which is located east-northeast and north of Webb Tract and Bacon Island, respectively.

A series of field investigations was conducted at Bouldin Island (HLA, 1992e). These field investigations consisted of 65 test borings, 169 cone penetration tests and 12 downhole geophysical surveys. Laboratory tests on soil samples recovered from the test borings were also performed to evaluate the static and dynamic soil characteristics. From these results, shear modulus reduction and damping curves were developed for the soils encountered in the borings. The downhole geophysical surveys also provided direct measurements of the in-situ shear wave velocities of the various foundation soils and levee materials. The results indicated a shear wave velocity range of 110-230 ft/sec for the soft peat.

A more recent study of the dynamic soil properties of the organic soil (peat) in the Delta Region was conducted by Boulanger et al (1997). Laboratory dynamic tests were performed on soil samples obtained in Shelby tubes on Sherman Island, which is located west and northwest of Webb Tract and Bacon Island, respectively. The resulting shear modulus reduction and damping curves for the peat were developed. A shear wave velocity range of 265-290 ft/sec was also obtained from testing the in-situ peat samples. These dynamic characteristics for the peat were shown to be consistent with those developed by other researchers for similar soils (Boulanger et al, 1998).

Figure A.1 compares the shear modulus reduction (G/G_{max}) and damping curves developed by HLA (1992e), and Boulanger et al (1997). The figure shows that the shear modulus decreases more rapidly with shear strain for the HLA (1991) model than the Boulanger, et al (1997) model. The damping curves are consistent for the two models, except for the larger shear strains (larger than about 1% shear strain) where HLA (1991) model gives higher damping values.

HLA (1990 and 1992e) identified three main seismic sources that control the seismic hazard at the Bouldin Island: 1) a magnitude 8.3 earthquake on the San Andreas fault that is capable of generating peak acceleration of 0.15g at the site; 2) a magnitude 7 earthquake on the Antioch fault that is capable of generating peak acceleration of 0.21g at the site; and 3) a magnitude 6.5 local earthquake that is capable of generating peak acceleration of 0.28g at the site.

A total of four earthquake time histories from past earthquakes were also selected for the dynamic response analyses (HLA, 1992e). These selected records were two rock motions recorded at U.C. Santa Cruz and Yerba Buena Island during the 1989 Loma Prieta earthquake and two artificial ground motion records developed by Seed et al (1972) for the San Andreas and Hayward fault events.

More recently (CALFED, 1999), probabilistic seismic hazard analysis and levee failure probabilistic evaluations were conducted for the Sacramento/San Joaquin Delta levees by the Seismic Vulnerability Sub-Team of CALFED's Levees and Channels Technical Team. In this study, the Delta Region was divided into four groups based on their expected seismic ground motions and the levee fragility to failure. Estimates for levee failure due to scenario earthquake events from nearby dominant seismic sources were also developed.

The results of the above probabilistic analysis indicate that local seismic sources in the Delta Region dominate the high frequency ground motions, including peak ground acceleration. Average peak accelerations at a 475-year return period of about 0.26g and 0.25g were calculated for Webb Tract and Bacon Island, respectively. The 475-year return period corresponds to about 10% probability of exceedance in 50 years. No information on the longer period ground motions was presented in the report, although the San Andreas and Hayward faults are expected to dominate the long period motions, and also are capable of longer duration.

Incidentally, the 475-year return period event is generally comparable to the deterministic ground motions obtained by HLA (1992e) using the MCE events on both local and distance sources.

Permanent deformations of the levee on the Bouldin Island were estimated using the dynamic soil characteristics and earthquake ground motions described above (HLA, 1990, 1991, 1992). The calculated deformations can be summarized as follows (HLA, 1992):

1. Deformations of up to about 7 feet are expected if the maximum credible earthquake were to occur at the end of levee construction.
2. Deformations of less than about 1 foot are expected if the maximum credible earthquake were to occur 5 years after construction.

A.3 EARTHQUAKE-INDUCED LEVEE DEFORMATIONS

A.3.1 Analysis Approach

The analysis approach taken for this study consisted of the following steps:

1. Select representative levee cross sections and material properties for analyses.
2. Develop design earthquake ground motions.
3. Compute the dynamic responses of the levee induced by the design earthquake motions.
4. Evaluate deformations of the levee based on the results of step 3.

The discussions of these analysis steps are given in the following subsections.

A.3.2 Select Cross Sections and Dynamic Soil Parameters

Four levee cross sections were selected for stability analyses for the two proposed reservoir islands, two cross sections for Webb Tract and two for Bacon Island. These cross sections were selected because they represent the critical sections along the levee axis. The cross sections selected for dynamic analyses are the same as those used in the static and pseudo-static slope stability analyses (see Section 3.3.2).

The nonlinear dynamic behavior of the levee and foundation materials was modeled using the equivalent-linear method proposed by Seed and Idriss (1970). In this method, the dynamic stress-strain behavior of soil is represented by that of a viscoelastic material with elastic modulus and viscous damping which are compatible with the amplitude of induced dynamic shear strain. The analysis is performed in iterations until the shear modulus and damping used in the analysis are compatible with the computed shear strains.

The parameters required for the viscoelastic soil model are the total unit weight, shear modulus (G), fraction of critical damping and Poisson's ratio. For sandy materials, the shear modulus at small strain (G_{max}) was assumed to depend on the effective confining pressure in accordance with the following equation, as proposed by Seed and Idriss (1970):

$$G_{max} = 1000 K_{2max} \sqrt{\sigma'_m}$$

where G_{max} = shear modulus at small strain in psf
σ'_m = mean effective confining pressure in psf
K_{2max} = a factor which depends on relative density, maximum particle size, gradation and other parameters.

The value of G_{max} and the variation of G/G_{max} with strain define the variation of shear modulus (G) with strain. The variation of G/G_{max} with strain is known as the modulus reduction curve. The mean effective confining pressure was computed using the following equation:

$$\sigma'_m = \frac{1+2K_0}{3} \sigma'_v$$

in which the σ_v' is the effective vertical pressure and K_0 is the coefficient of earth pressure at rest.

For clayey soils, shear modulus was estimated from the shear wave velocity using the following equation:

$$G_{max} = \rho V_s^2$$

where ρ and V_s are the soil density and shear wave velocity, respectively.

Table A.1 presents the selected dynamic soil parameters used in the analyses. The unit weights are those used in the static analyses. The values of K_{2max} and V_s selected for the various foundation and levee materials were estimated based on the results of previous studies (HLA, 1991, 1992 and Boulanger, 1997), experience with similar soil conditions and engineering judgment.

Table A.1 also lists the damping and shear modulus reduction curves used for each levee and foundation materials. The shear modulus reduction and damping curves selected for the clayey soils were those developed by Vucetic and Dobry (1991) for clayey soils with Plasticity Index (PI) of about 50 and 100. Shear modulus reduction and damping curves (mean curves) developed by Seed and Idriss (1970) for sand were used for the sandy soils. For peat, we used the relationships developed by Boulanger (1997) for Sherman Island peat. The selected modulus reduction and damping curves are shown in Figure A.2, as a function of shear strain.

It was expected that the levee materials will control the overall dynamic behavior of the levee. Accordingly, for the dynamic response analysis, the underlying clay deposit was modeled as an elastic half space in order to properly account for its energy transmitting characteristics.

A.3.3 Design Earthquake Ground Motions

The approach used to develop the design earthquake ground motions can be summarized as follows:

1. Select an appropriate hazard exposure level for design and develop the design response spectrum consistent with the selected hazard exposure level.
2. Select earthquake acceleration time histories for input motions.
3. Spectrally modify selected acceleration time histories to match the design response spectrum developed in step 1.

These steps are discussed below, and the spectrally matched ground motions were used in the dynamic response analysis, as described in Section A.3.4.

Design Response Spectrum

We selected a hazard exposure level that corresponds to a 10% probability of exceedance in 50 years for the design. Assuming that earthquake occurrences follow a Poisson process, a 10% probability of exceedance in 50 years results in a return period of about 475 years (or annual frequency of occurrence of 2.1×10^{-03}). The 475-year return period hazard exposure level is consistent with the requirement adopted by the 1997 Uniform Building Code (UBC) and was used in a previous study conducted for the project (HLA, 1990).

A site-specific probabilistic seismic hazard analysis is beyond the scope of services of our current study. We have, therefore, developed the design response spectrum using the results of previous studies (CALFED, 1999; HLA, 1992e), supplemented by current understanding of regional seismic sources and the deterministic ground motion analysis. In developing the design response spectrum, we used the results of the recent probabilistic seismic hazard analysis conducted by the Seismic Vulnerability Sub-Team of CALFED's levees and channels Technical Team (1999) to identify controlling seismic sources. Accordingly, we considered two controlling seismic source groups: local and distant source groups. The local source group represents the seismic sources in the Delta Region while the distant source group represents the San Andreas and Hayward faults.

The results of the CALFED study indicate that the local seismic source group can be represented by an earthquake of magnitude (M_w) 6 at a distance of about 20 km, for the 475-year return period hazard exposure level. For the San Andreas and Hayward faults, earthquakes with M_w of 7.7 at a distance of about 85 km and M_w of 7.2 at a distance of about 56 km were used, respectively. The magnitudes of the San Andreas and Hayward faults were selected based on the current understanding of the fault characteristics that are consistent with the 475-year return period earthquake magnitudes (USGS Working Group of 1999). The peak accelerations corresponding to the local, San Andreas and Hayward seismic events were set at 0.26g, 0.13g and 0.11g, respectively, based on the results of the CALFED study.

Figure A.3 shows the 5% damped response spectra calculated for these controlling events, scaled to their respective peak accelerations, using the ground motion attenuation relationships developed by Abrahamson and Silva (1997) and Sadigh et al (1997) for stiff soils. The design response spectrum was then developed by smoothing and enveloping these response spectra, as shown in Figure A.3. The design response spectrum is applicable for a free-field stiff soil site condition with 5% damping ratio.

Selected Earthquake Time Histories

Two horizontal earthquake acceleration time histories recorded during past earthquakes were selected for the dynamic analysis. These records were from the 1992 Landers earthquake with M_w of 7.3, recorded at Fort Irwin station (station #24577), and the 1987 Whittier Narrows earthquake with M_w of 6, recorded at Altadena, Eaton Canyon station (station #24402). Table A.2 lists these selected motions along with their closest distances from the rupture planes and recorded peak accelerations. The site conditions at these recording stations are classified as stiff soil sites. Figures A.4 and A.5 show the time history plots of the acceleration, velocity and displacement of these selected earthquake time histories.

The record from the 1992 Landers earthquake was selected to represent the larger and more distant earthquakes on the San Andreas and Hayward faults. The 1987 Whittier Narrows earthquake was selected to represent the local seismic source group.

Spectrally Matched Earthquake Time Histories

The response spectral values calculated from the selected acceleration time histories (natural time histories) have peaks and valleys that deviate from the design response spectrum (target response spectrum). To develop acceleration time histories with overall characteristics that match the target response spectrum, modifications to the natural time histories were necessary.

The two pairs of selected acceleration time histories were spectrally matched to the design response spectrum using the method proposed by Lilhanand and Tseng (1988) and modified by Abrahamson (1993). The plots of the acceleration, velocity and displacement time histories of these spectrally matched motions are presented in Figures A.6 and A.7. The 5% damped response spectra for the natural and modified motions are shown in Figures A.8 and A.9 along with the target spectrum. It can be seen from these figures that the response spectra calculated from the modified time histories closely match the target spectrum and the general characteristics of the modified time histories resemble those of the natural motions.

A.3.4 Dynamic Response Analysis

The analysis for the levee response under the earthquake loads was carried out using the computer program QUAD4M (Hudson et al, 1994). QUAD4M is a two-dimensional plan-strain, finite element code for dynamic analysis. This program uses equivalent linear stress-strain relationships for soils. The program also uses a time domain integration scheme that allows the user to reassign different material properties at any time during the seismic shaking. QUAD4M incorporates a compliant base (energy-transmitting base) which can be used to model the elastic half-space.

The finite element models used for the dynamic analyses are shown in Figures A.10, A.15, A.20 and A.25 for the four selected levee cross sections. Compliant bases were used at the bottom of the finite element models to prevent total reflection of wave energy at the fixed bases. The shear wave velocity for the underlying elastic half space was taken equal to that of the clay deposit beneath the sand layer. Earthquake acceleration time histories were input to the finite element models at the base of the sand layer (i.e., at the interface between sand layer and clay deposit). These input acceleration time histories were obtained by deconvolving the spectrally matched time histories to an elevation corresponding to the base of the sand layer. We used the one-dimensional wave propagation computer program SHAKE (Schnabel et al, 1972) to deconvolve the ground motions.

Our review of the available subsurface data indicates that the levee materials and foundation soils are not susceptible to widespread liquefaction under the design earthquake ground motions (see Section A.4). Pockets of loose sand deposit exist within the levee and foundation soils. The data on subsurface soils, however, indicate that these loose sand pockets are limited in extent. Therefore, we do not expect that during the occurrence of the design earthquake significant liquefaction, and hence significant changes in dynamic soil properties and levee responses, will occur. As such, in carrying out the analyses, we did not account for the effects of softening (or reduction in shear modulus) of the sandy soils.

The dynamic response analyses were carried out to compute the average horizontal acceleration (K_{ave}) time histories of the potential (critical) slide masses within the levee. These critical slide masses were identified through the static slope stability analyses, as described in Section 3.5.

The critical slide masses analyzed for the dynamic responses are presented in Figures 3.5.25 and 3.5.26 for the Webb Tract levee at Station 160+00, Figures 3.5.7 and 3.5.8 for the Webb Tract levee at Station 630+00, Figures 3.5.34 and 3.5.35 for the Bacon Island levee at Station 25+00, and Figures 3.5.43 and 3.5.44 for the Bacon Island levee at Station 265+00. It is noted that the critical slide masses on the slough-side slopes were identified using groundwater conditions different from those used to identify critical slide masses on the reservoir-side slopes (see Section 3.3.3). The average horizontal acceleration was calculated by computing the dynamic response of the levee to the design earthquake ground motions and averaging various stresses within or close to the sliding surface.

Figures A.11 and A.12 show the computed average horizontal accelerations (K_{ave}) for the critical slide masses of the levee cross section at Station 160+00 of Webb Tract. Figures A.16 and A.17 show the computed average horizontal accelerations (K_{ave}) for the critical slide masses of the levee cross section at Station 630+00 of Webb Tract. Figures A.21 and A.22 show the computed average horizontal accelerations (K_{ave}) for the critical slide masses of the levee cross section at Station 25+00 of Bacon Island. Figures A.26 and A.27 show the computed average horizontal accelerations (K_{ave}) for the critical slide masses of the levee cross section at Station 265+00 of Bacon Island.

The peak average horizontal accelerations (K_{max}) for these critical masses were tabulated in Table A.3. These peak values will be used for estimating the levee deformations using the simplified method of Makdisi and Seed (1978), as discussed below.

A.3.5 Earthquake-induced Levee Deformations

Seismic-induced permanent deformations of the levee were estimated using both the Newmark Double Integration Method (1965) and the Makdisi and Seed Simplified Procedure (1978).

The Newmark Double Integration Method (1965) is based on the concept that deformations of an embankment will result from incremental sliding during the short periods when earthquake inertia forces in the critical slide mass exceed the available resisting forces. This method involves the calculation of the displacement (deformation) increment of a critical slide mass at each time step using the average horizontal acceleration (K_{ave}) and the value of yield acceleration (K_y) calculated for the slide mass. The displacement increment is calculated by double integrating the difference between K_{ave} and K_y values at time acting on the slide mass over time. The estimated permanent deformation of the slide mass is then taken as the sum of displacement increments at the end of ground shaking.

The Newmark method assumes that a well-defined failure surface will develop and that the materials will exhibit elastic-plastic behavior. Although these assumptions are only rough approximation on the true behavior of the slide mass under the earthquake shaking, the method has been shown to provide good estimates of the observed earthquake-induced deformations of dams (Makdisi and Seed, 1978).

Figures A.13 and A.14 show the computed permanent deformations for the critical slide masses of the levee cross section at Station 160+00 of Webb Tract. Figures A.18 and A.19 show the computed permanent deformations for the critical slide masses of the levee cross section at Station 630+00 of Webb Tract. Figures A.23 and A.24 show the computed permanent deformations for the critical slide masses of the levee cross section at Station 25+00 of Bacon

Island. Figures A.28 and A.29 show the computed permanent deformations for the critical slide masses of the levee cross section at Station 265+00 of Bacon Island. In these figures, we show the deformations calculated using the K_{ave} time histories applied in the "normal" and "reversed" directions. The "reserved" direction was obtained by flipping the time history about the time axis.

The simplified procedure of Makdisi and Seed (1978) was developed based on observations on dam performance during past earthquakes and analysis results. In this method, the inertia forces on the slide mass are represented by the peak average horizontal acceleration (K_{max}) induced by the design earthquake. Empirical relationships relating the ratio of the K_y and K_{max} (K_y/K_{max}) and the average deformation were then used to estimate the levee deformation.

The calculated deformations of the identified critical slide masses of the levees on the Webb Tract and Bacon Island are tabulated in Table A.3. Deformations estimated using the Newmark Double Integration Method and the simplified procedure of Makdisi and Seed are both listed for comparison. In estimating the deformation, we rounded up the calculated deformation to the nearest 0.5 ft. Also, the empirical relationships of Makdisi and Seed (1978) were developed for a magnitude range of 6.5 to 8.25. Since the 1987 Whittier Narrows earthquake had a magnitude of 6.0, we used the empirical relationships developed for magnitude 6.5 to estimate the deformations due to the 1987 Whittier Narrows earthquake.

Maximum calculated deformations are about 3-4 feet for the slough-side slopes. On the reservoir side, slope deformations of about 1.5-3.5 feet were estimated. The smaller deformations for the reservoir-side slopes are due to the increased stability provided by the proposed new fills. Both Newmark Double Integration Method and Makdisi and Seed simplified procedure give comparable results.

A.4 SEISMIC-INDUCED GEOLOGIC HAZARDS

A.4.1 Liquefaction

A liquefaction susceptibility evaluation was performed for Webb Tract and Bacon Island. We used the SPT blow counts from the exploratory borings to assess the potential for liquefaction during the occurrence of the design ground motions. The evaluation procedure for liquefaction susceptibility proposed by the NCEER Workshop (Youd and Idriss,1997) was utilized for this study.

Penetration blow counts were taken from the boring logs presented in a report by Harding Lawson Associates (HLA, 1989). We applied the corrections for the fines contents and overburden pressure to the measured blow counts. No corrections for the drilling procedure and testing equipment were applied due to the lack of specific details on equipment and drilling techniques used.

Two design ground motion criteria were selected for the analyses: earthquakes with magnitude (M_w) 6 and peak ground acceleration of 0.25g, and magnitude (M_w) 7.7 and peak ground acceleration of 0.13g. These ground motions represent the controlling events for the local and distant seismic sources and are consistent with those used in the dynamic response analyses, as described in Section A.3.

The results of the analyses indicate that a few pockets of potentially liquefiable soil deposit exist in the levees and foundation soils. We believe, however, that these liquefiable soil pockets are confined in limited areas and therefore are expected to have negligible adverse effects on the stability of the levees.

A.4.2 Loss of Bearing Capacity

Seismic-induced bearing capacity reduction is associated mainly with the occurrence of liquefaction or pore water pressure generation. The reduction may be substantial for shallow foundations supported on or near the liquefied soils. Based on the results of liquefaction evaluation and the absence of shallow liquifiable foundations layers at the project site, we judge that the risk of loss of bearing capacity that may affect levee performance is insignificant.

A.4.3 Dynamic Soil Compaction

Similar to the seismic-induced bearing capacity loss, the dynamic soil compaction would only be significant following the occurrence of extensive liquefaction and associated liquefaction-induced settlement. Since the potential for liquefaction at the project islands is limited to few isolated pockets, we judge that the potential for dynamic soil compaction (settlement) at these islands is negligible.

A.4.4 Levee Overtopping During Seismic-Induced Seiche

Earthquakes can cause overtopping of levees due to three primary mechanisms: landslide generated waves, static displacement of the reservoir or dynamic displacement of the reservoir. Both the landslide induced waves and static displacement of the reservoir are not expected to occur at the project reservoirs.

Records for past occurrences of seiche are generally incomplete. The largest seiche reported in the United States was in Lake Ouachita in Arkansas with a maximum amplitude of about 0.44 m (1.5 feet). We have estimated the amplitudes of seismic-induced waves (dynamic displacement of reservoir water) using the procedure of United States Committee on Large Dams (USCOLD, 1995). The results of analysis seem to indicate a negligible amplitude of seismic-induced wave (less than 1 foot). It should be noted that this procedure was developed for a limited body of water (tanks, dams) and has been assumed to be applicable to the DW project reservoirs.

Table A.1
Dynamic Soil Parameters Used in the Response Analysis

Description	Moist Unit Weight pcf	K_{2max}	Shear Wave Velocity ft/sec	Modulus and Damping Curves
Levee Materials				
New fills: sand	120	80	-	Sand[3]
Fills: sand	105-110	25	-	Sand[3]
Fills: soft clay	70	-	250	Clay[1]
Fills: silty sand with fat clay	110	25	-	Sand[3]
Fills: clay with peat	80	-	300	Clay[1]
Fills: silty clay with sand	110		450	Clay[2]
Peat	70	-	250	Peat[4]
Foundation Materials				
Sand	120-125	80	-	Sand[3]
Clay	127	-	700	Clay[2]

Note : 1: Relationships of Vucetic and Dobry (1991) for PI = 100
2: Relationships of Vucetic and Dobry (1991) for PI = 50
3: Relationships of Seed and Idriss (1970) for mean
4: Relationships of Boulanger et al (1997)

Table A.2
Summary of the Earthquake Records Used in the Dynamic Response Analysis

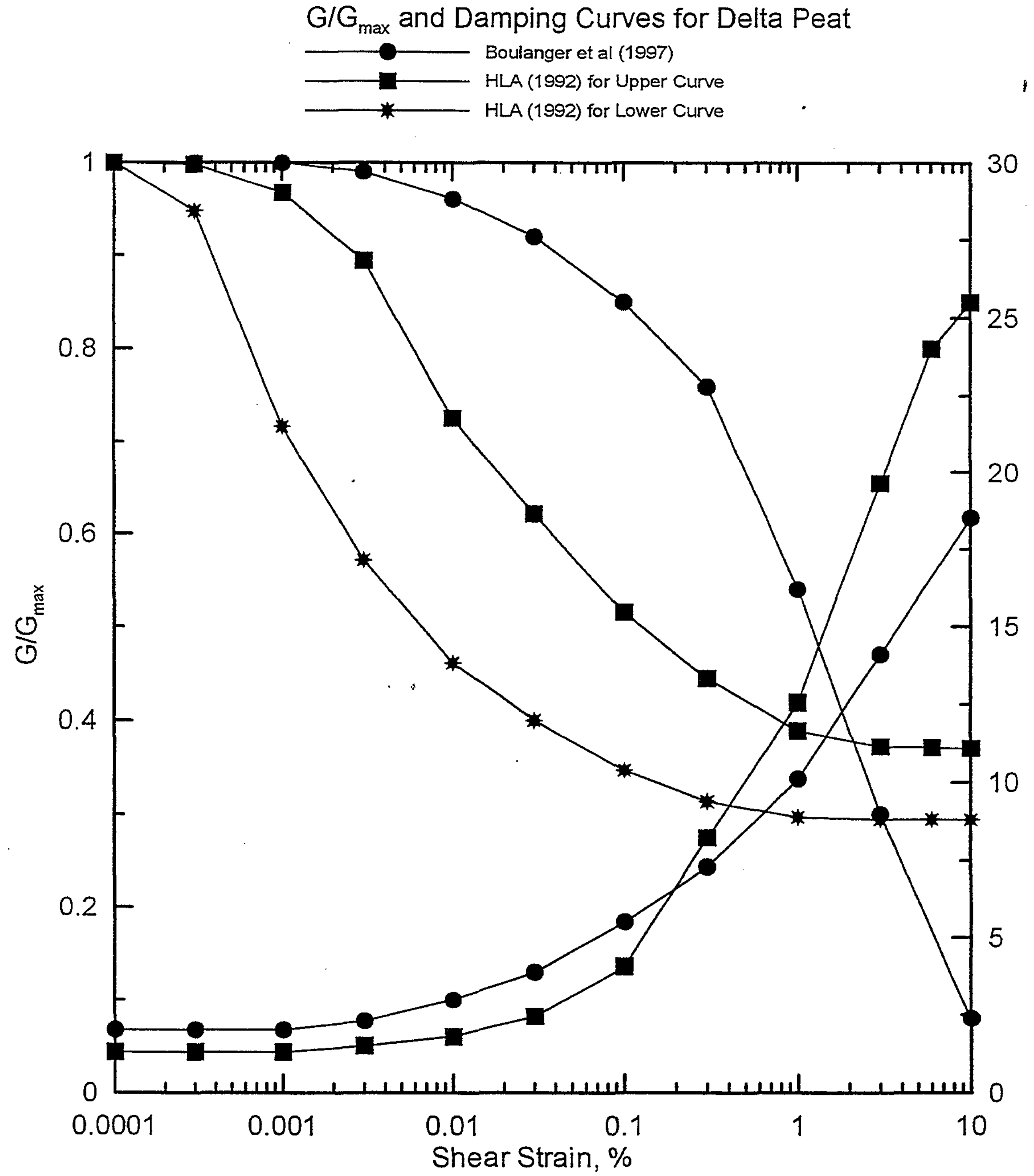
Earthquake	M_w	Recording Station			Comp	Recorded PGA (g)
		Distance (km)	Station #	Site Condition		
1987 Whittier Narrows	6.0	18	24402[b]	Soil[a]	90°	0.15
1992 Landers	7.3	64	24577[c]	Soil[a]	0°	0.11

Note : a = Deep Stiff Soil Site
b = Altadena – Eaton Canyon Station
c = Fort Irwin Station

Table A.3
Calculated Seismic-induced Slope Deformations

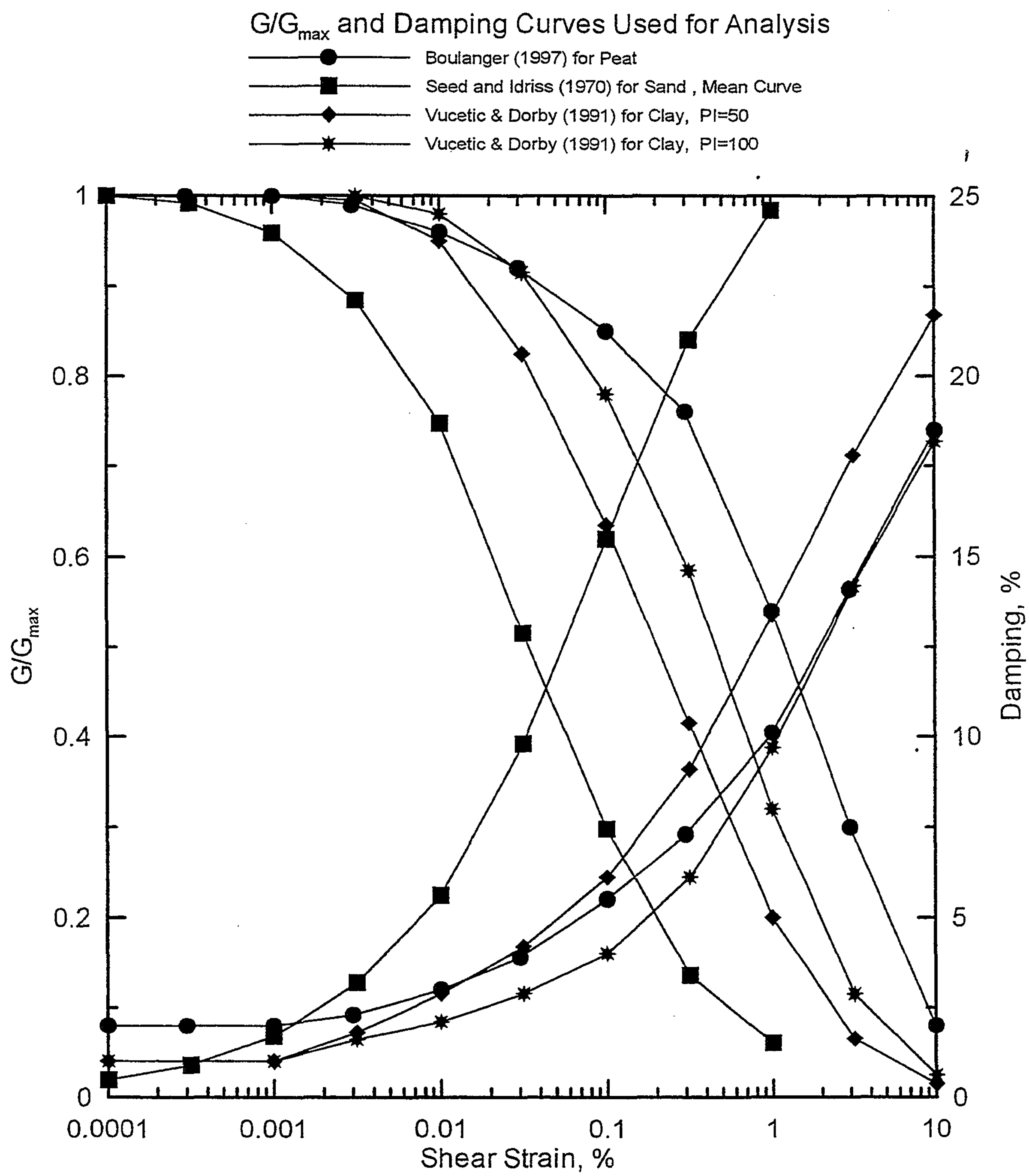
Critical Slide	K_y (g)	K_{max} (g)	Max Deformation (ft)	
			Newmark[1]	Makdisi & Seed[2]
Webb Tract at St. 160+00				
Reservoir-side Slope				
Crest Slide[3]	0.114	0.40	2.0	0.5-1.5
Slough-side Slope				
Crest Slide[3]	0.025	0.21	3.5	0.5-3.5
Webb Tract at St. 630+00				
Reservoir-side Slope				
Crest Slide[4]	0.151	0.36	1.5	0-1.0
Slough-side Slope				
Crest Slide[4]	0.027	0.26	4.0	1.0-4.0
Bacon Island at St. 25+00				
Reservoir-side Slope				
Upper Toe Slide[5]	0.148	0.47	3.5	0.5-1.0
Slough-side Slope				
Crest Slide[5]	0.048	0.31	3.5	0.5-3.0
Bacon Island at St. 265+00				
Reservoir-side Slope				
Crest Slide[6]	0.133	0.36	1.5	0.5-1.0
Slough-side Slope				
Crest Slide[6]	0.0385	0.28	3.5	0.5-3.0

Note:
1: Newmark Double Integration Method (1965)
2: Makdisi and Seed Simplified Method (1978)
3: Refer to Figures 3.5.25 and 3.5.26.
4: Refer to Figures 3.5.7 and 3.5.8.
5: Refer to Figures 3.5.34 and 3.5.35.
6: Refer to Figures 3.5.43 and 3.5.44.



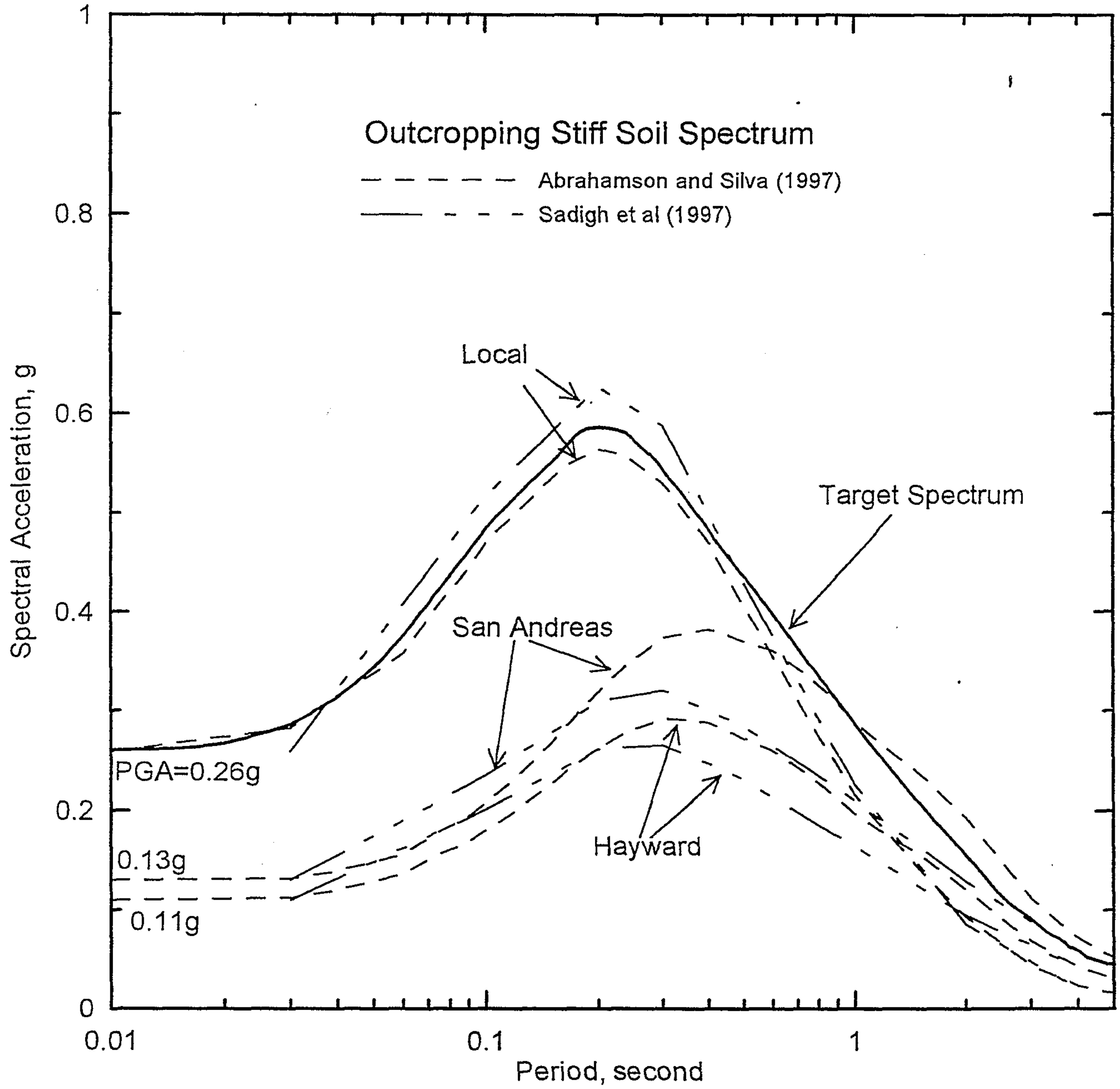
Project No. 410709903000	Delta Wetlands	Comparison of Shear Modulus Reduction and Damping Curves for Delta Peat	Figure A.1
URS Greiner Woodward Clyde			

C-063545



Project No. 410709903000	Delta Wetlands	Shear Modulus and Damping Curves Used for Dynamic Response Analysis	Figure A.2
URS Greiner Woodward Clyde			

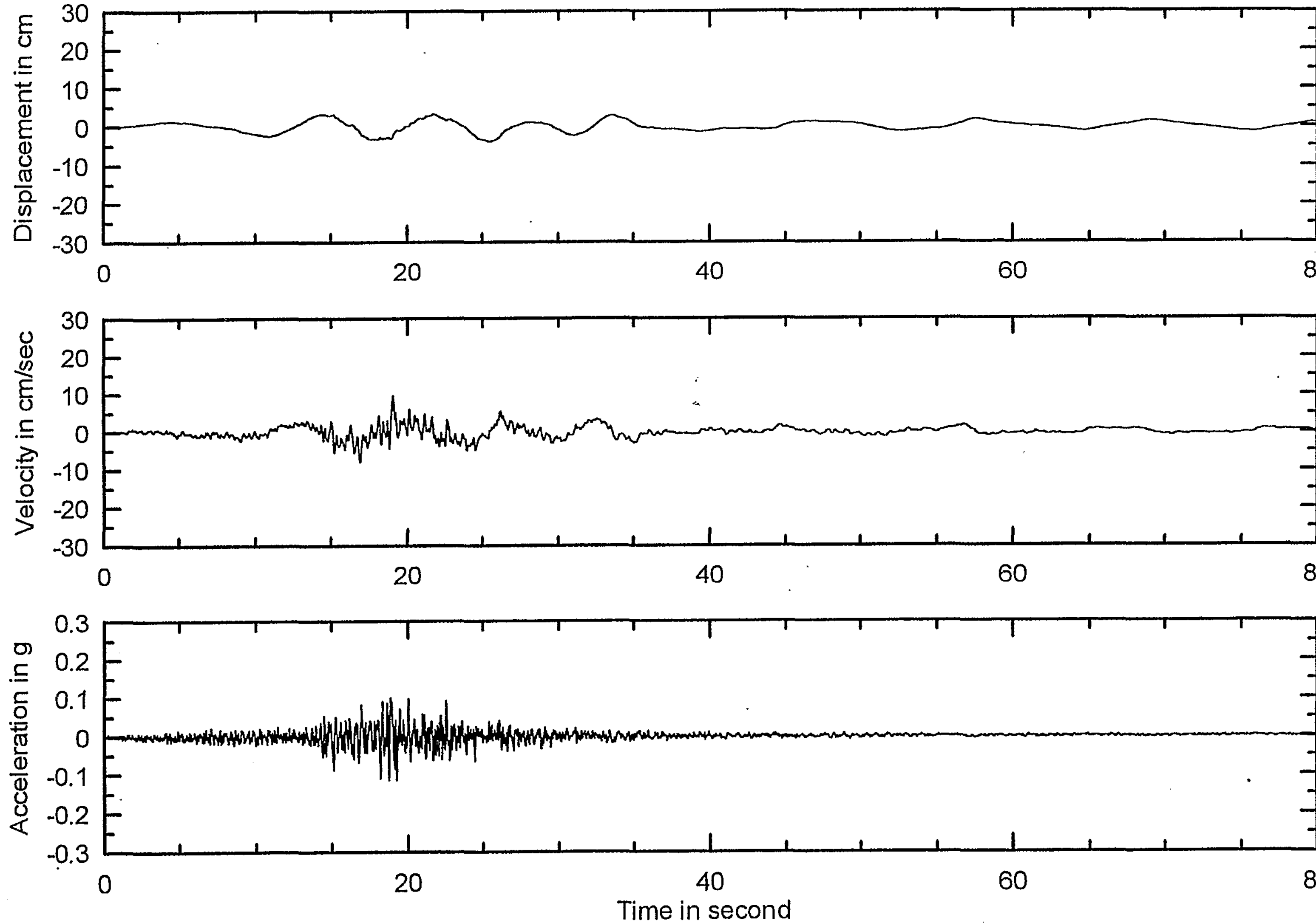
C-063546



Project No. 410709903000	Delta Wetlands	Design Response Spectrum for Webb Tract and Bacon Island	Figure A.3
URS Greiner Woodward Clyde			

C-063547

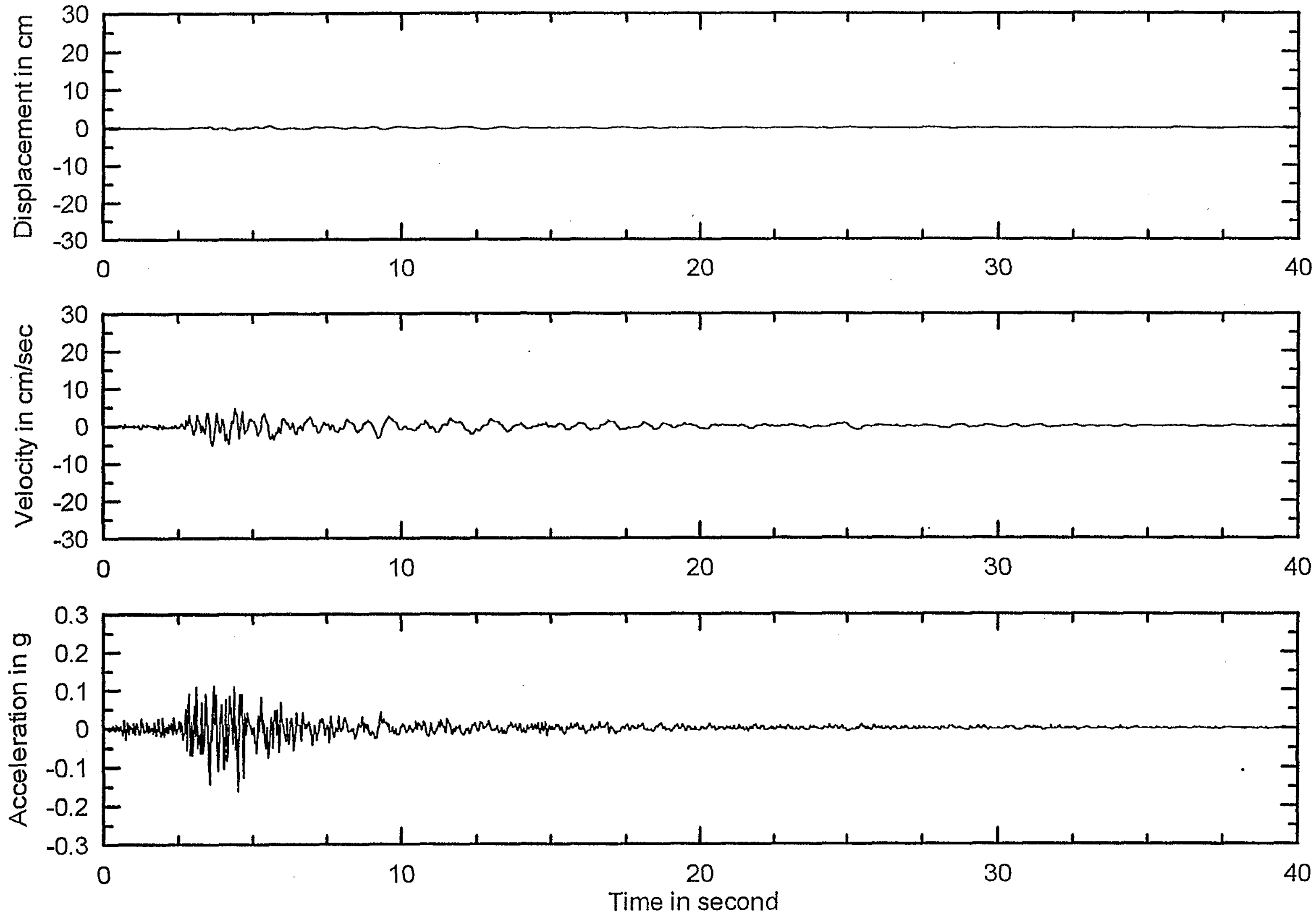
1992 Landers Earthquake at St. 24577 (Fort Irwin), 0 Degree Component
Recorded Ground Motions



Project No. 410709903000	Delta Wetlands	Recorded Ground Motions for 1992 Landers Earthquake at St. – 24577, 0 Degree Component	Figure A.4
URS Greiner Woodward Clyde			

C-063548

1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon), 90 Degree Component
Recorded Ground Motions



PROJECT NO. 410709903000	Delta Wetlands	Recorded Ground Motions for 1987 Whittier Narrows Earthquake at St. 24402, 90 Degree Component	Figure A.5
URS Greiner Woodward Clyde			

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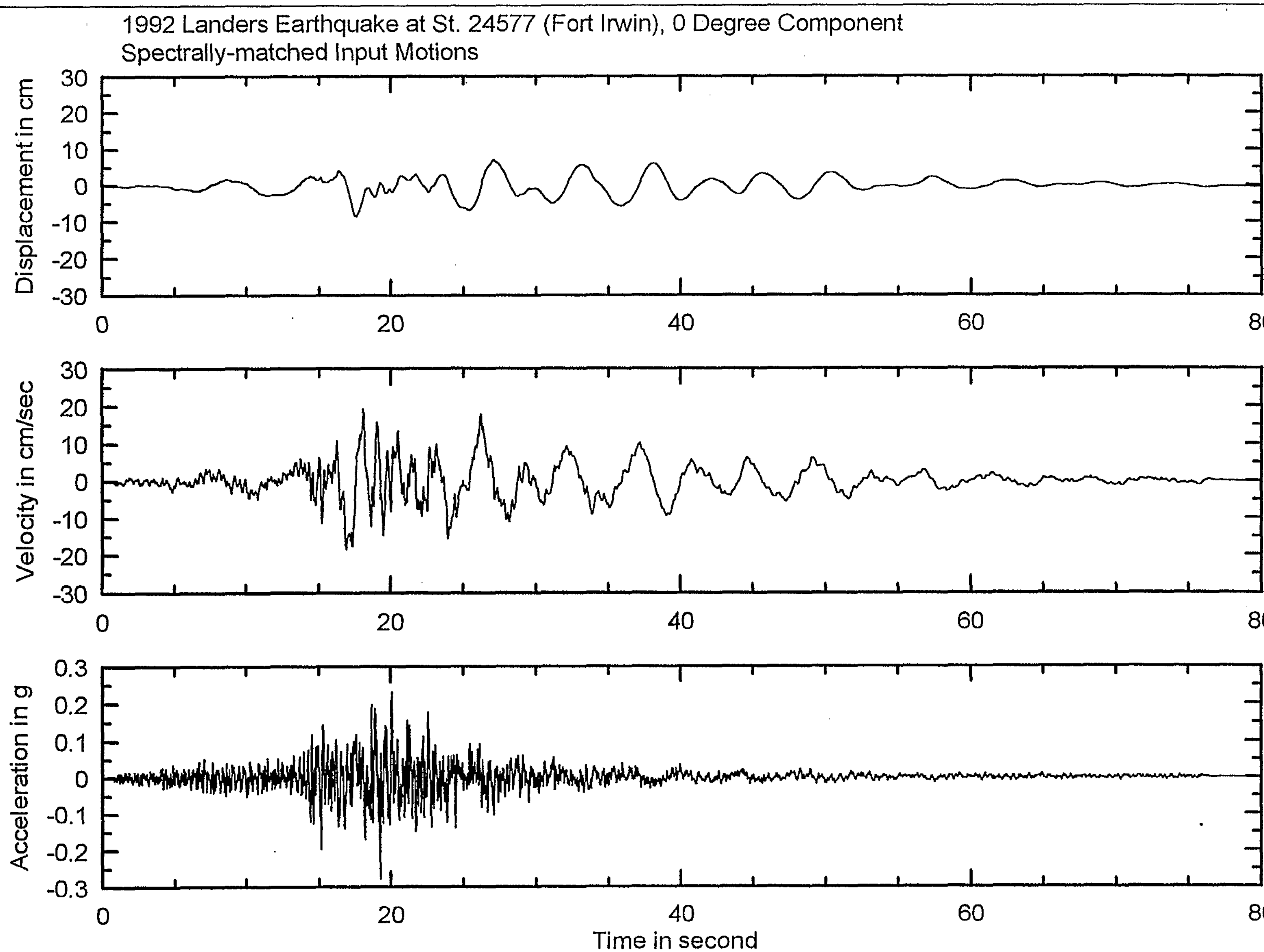
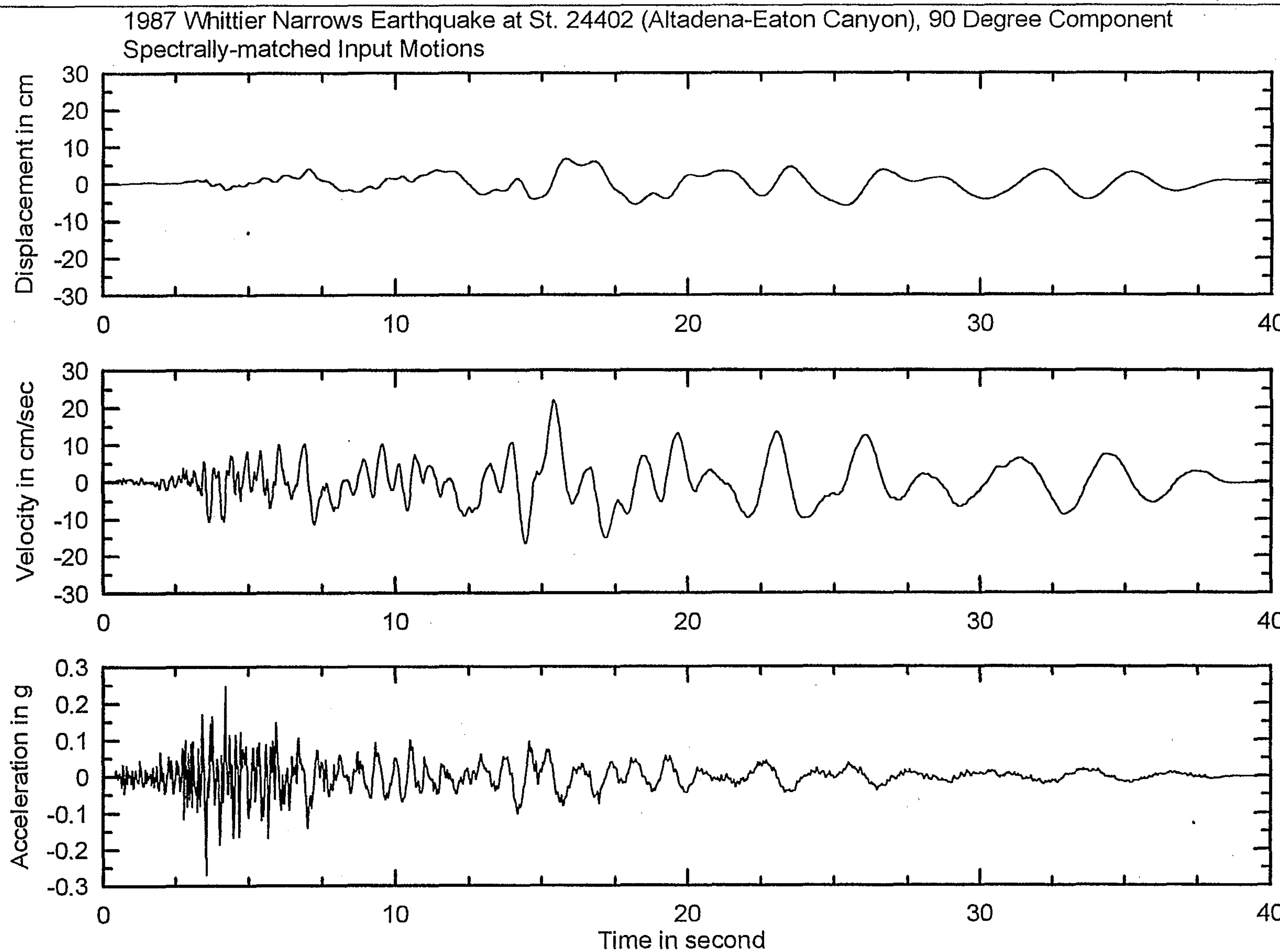
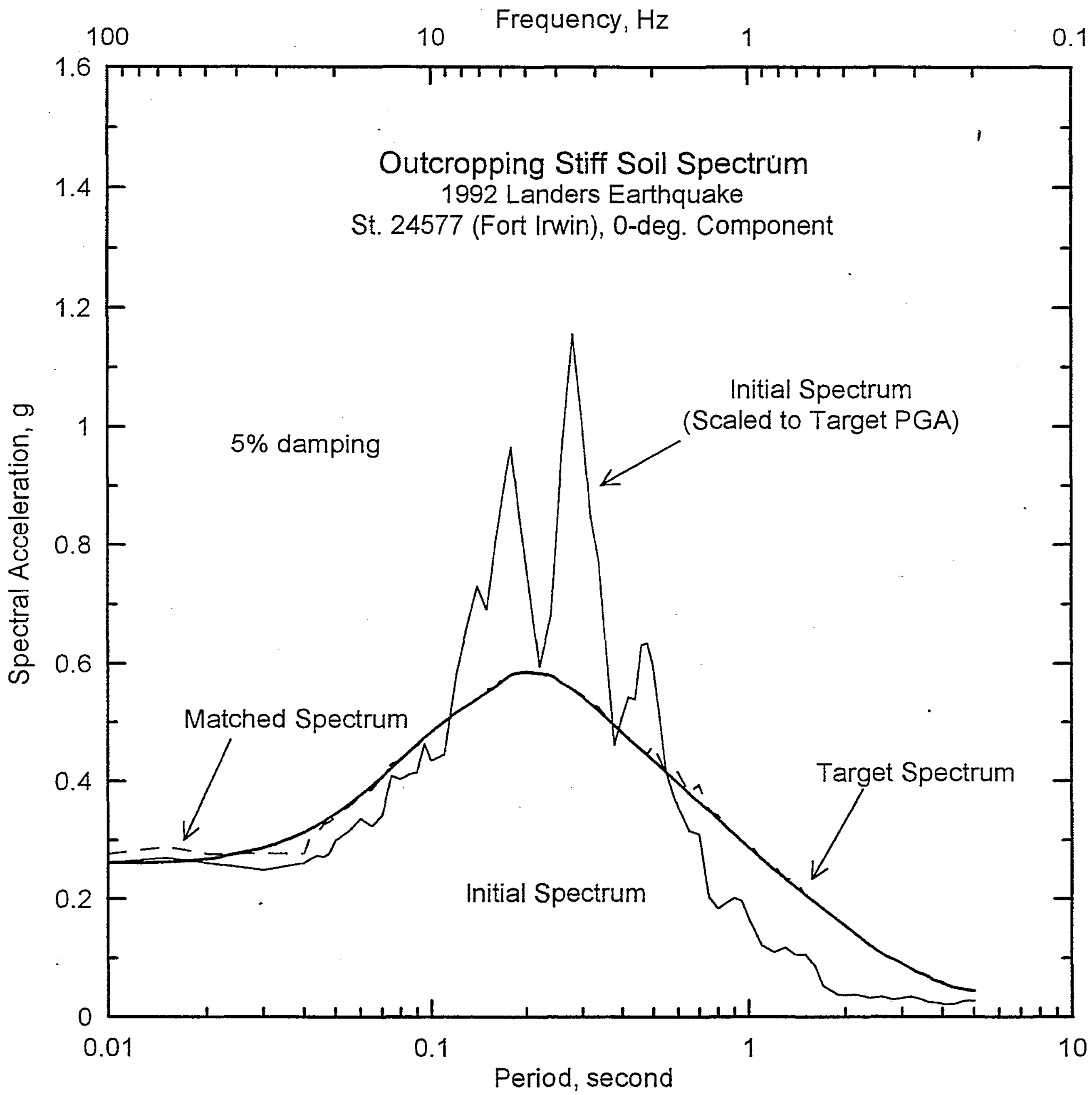


Project No. 410709903000	Delta Wetlands	Spectrally Matched Ground Motions for 1992 Landers Earthquake at St. 24577, 0 Degree Component	Figure A.6
URS Greiner Woodward Clyde			

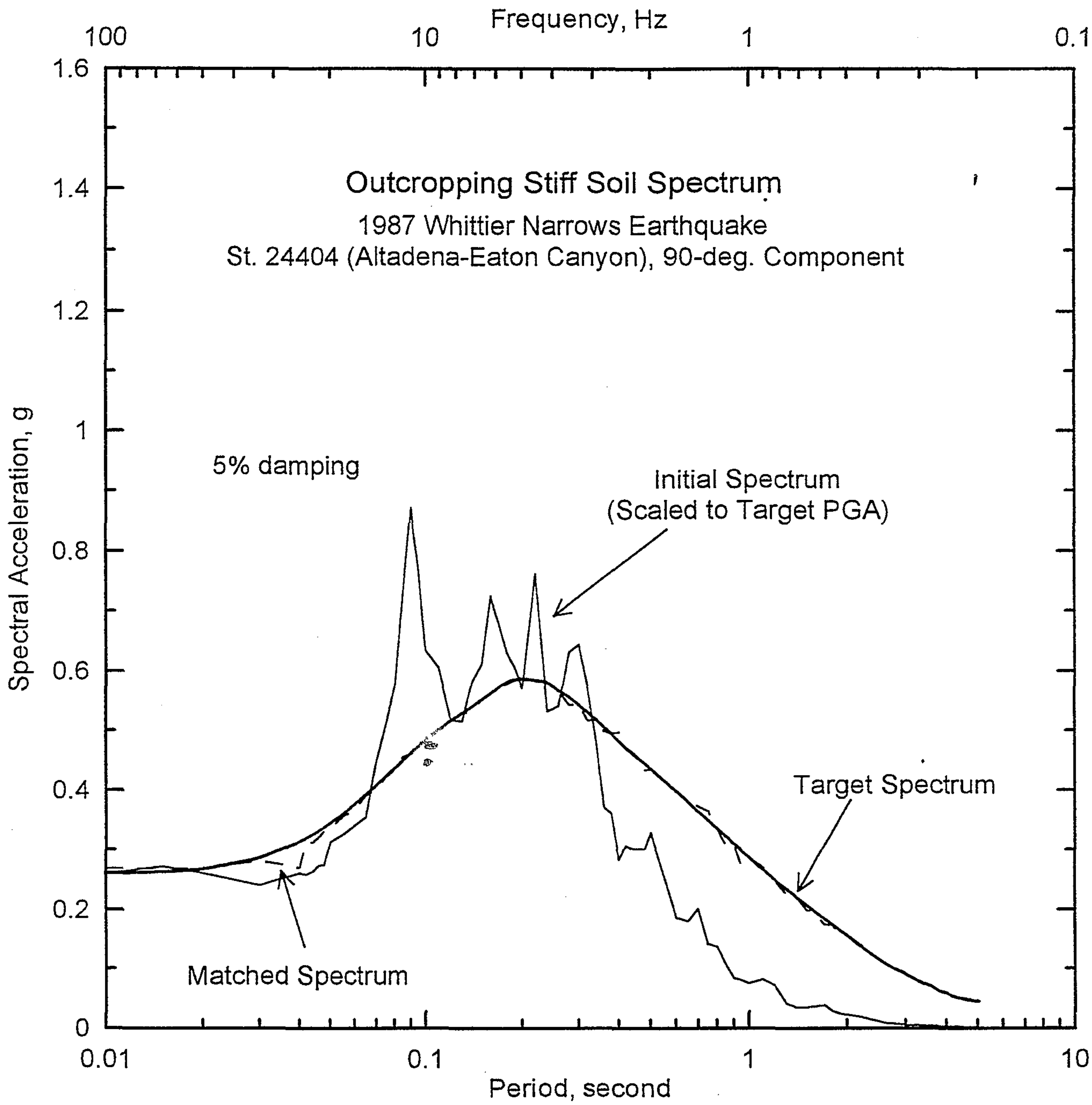


Project No. 410709903000	Delta Wetlands	Spectrally Matched Ground Motions for 1987 Whittier Narrows Earthquake at St. 24402, 90 Degree Component	Figure A.7
URS Greiner Woodward Clyde			

C-063551



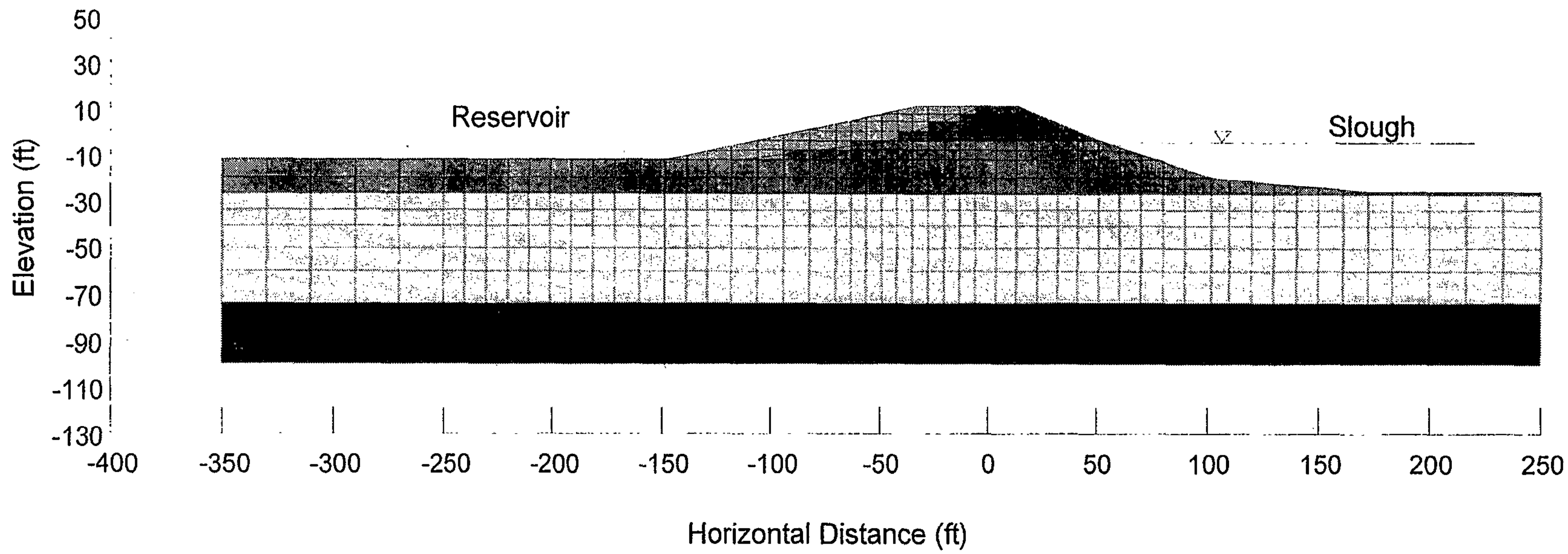
Project No. 410709903000	Delta Wetlands	Design Response Spectrum and Recorded and Modified Spectra for 1992 Landers Earthquake	Figure A.8
URS Greiner Woodward Clyde			



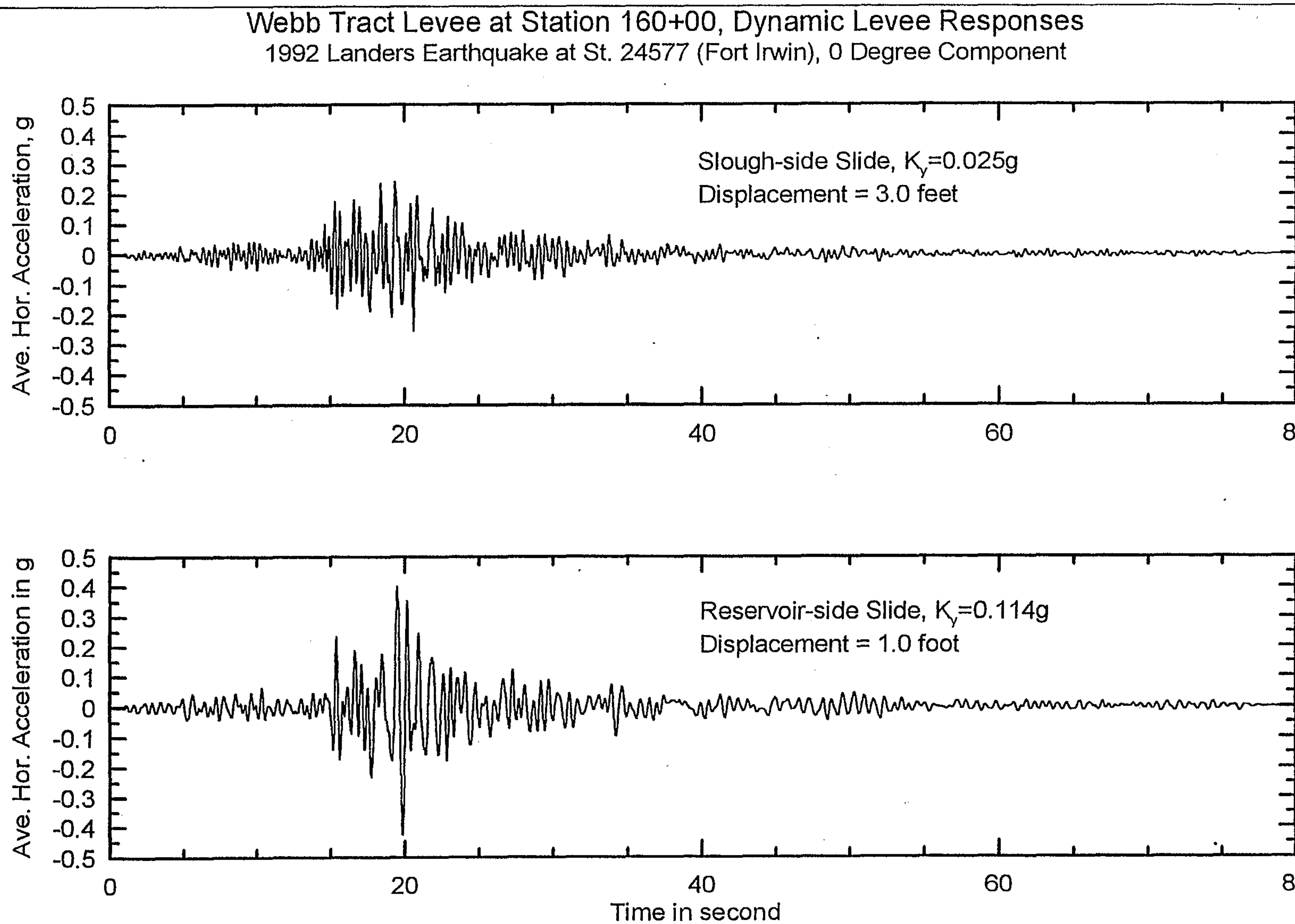
Project No. 410709903000	Delta Wetlands	Design Response Spectrum and Recorded and Modified Spectra for 1987 Whittier Narrows Earthquake	Figure A.9
URS Greiner Woodward Clyde			

C-063553

Webb Tract At Station 160+00
Final Configuration Cross Section



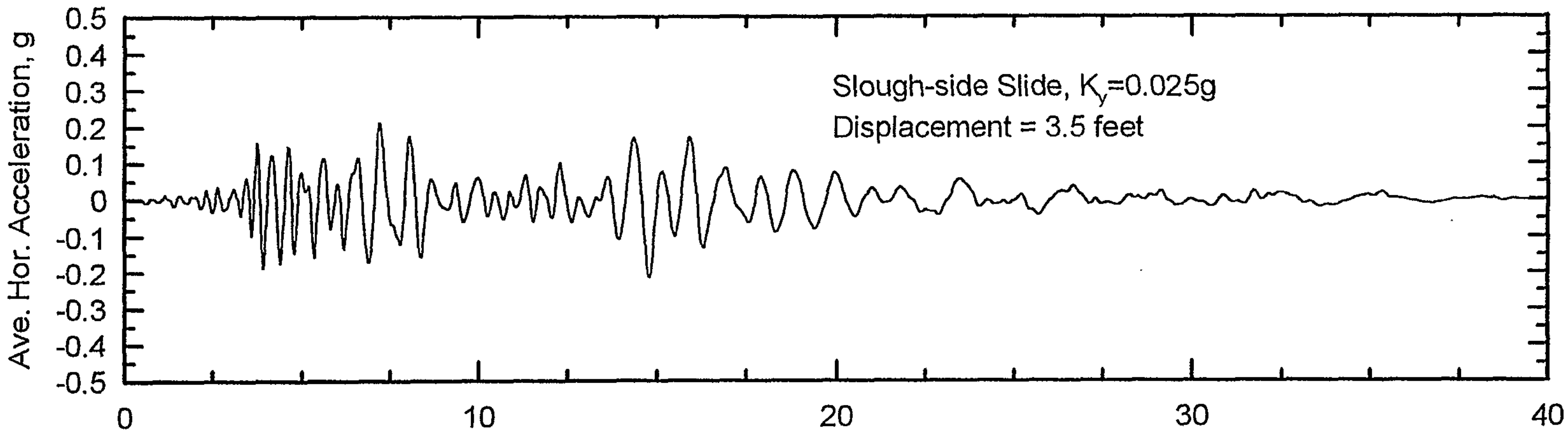
Project No. 410709903000	**Delta Wetlands**	Finite Element Mesh for Webb Tract Levee at Station 160+00	Figure A.10
URS Greiner Woodward Clyde			

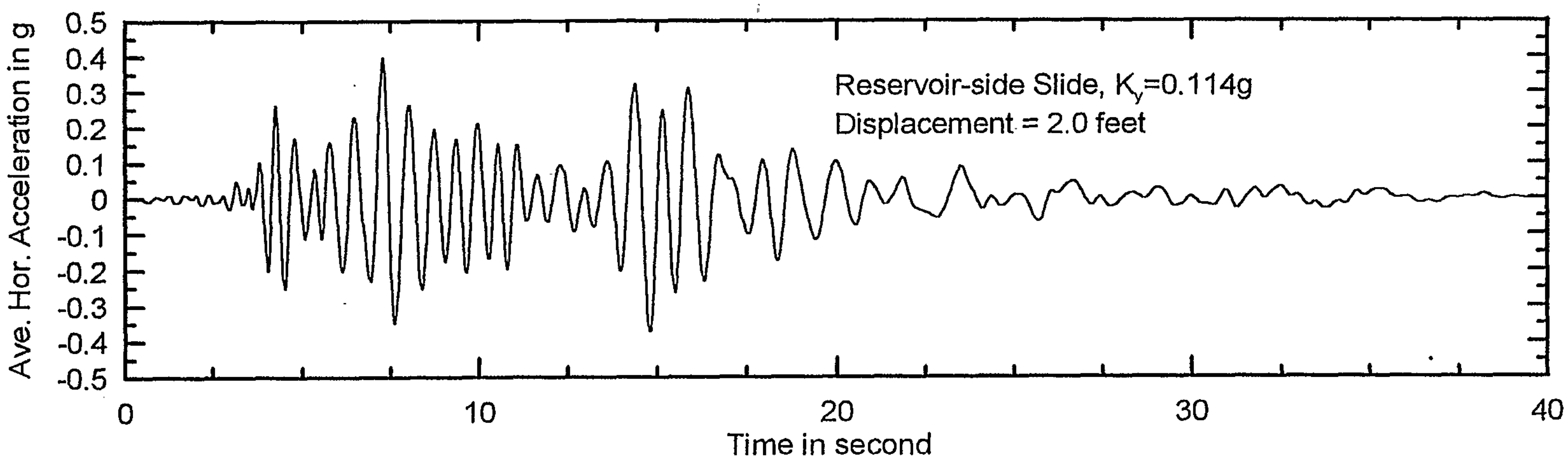


Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1992 Landers Earthquake - Webb Tract Levee at St. 160+00	Figure A.11
URS Greiner Woodward Clyde			

Webb Tract Levee at Station 160+00, Dynamic Levee Responses

1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon),90 Degree Component

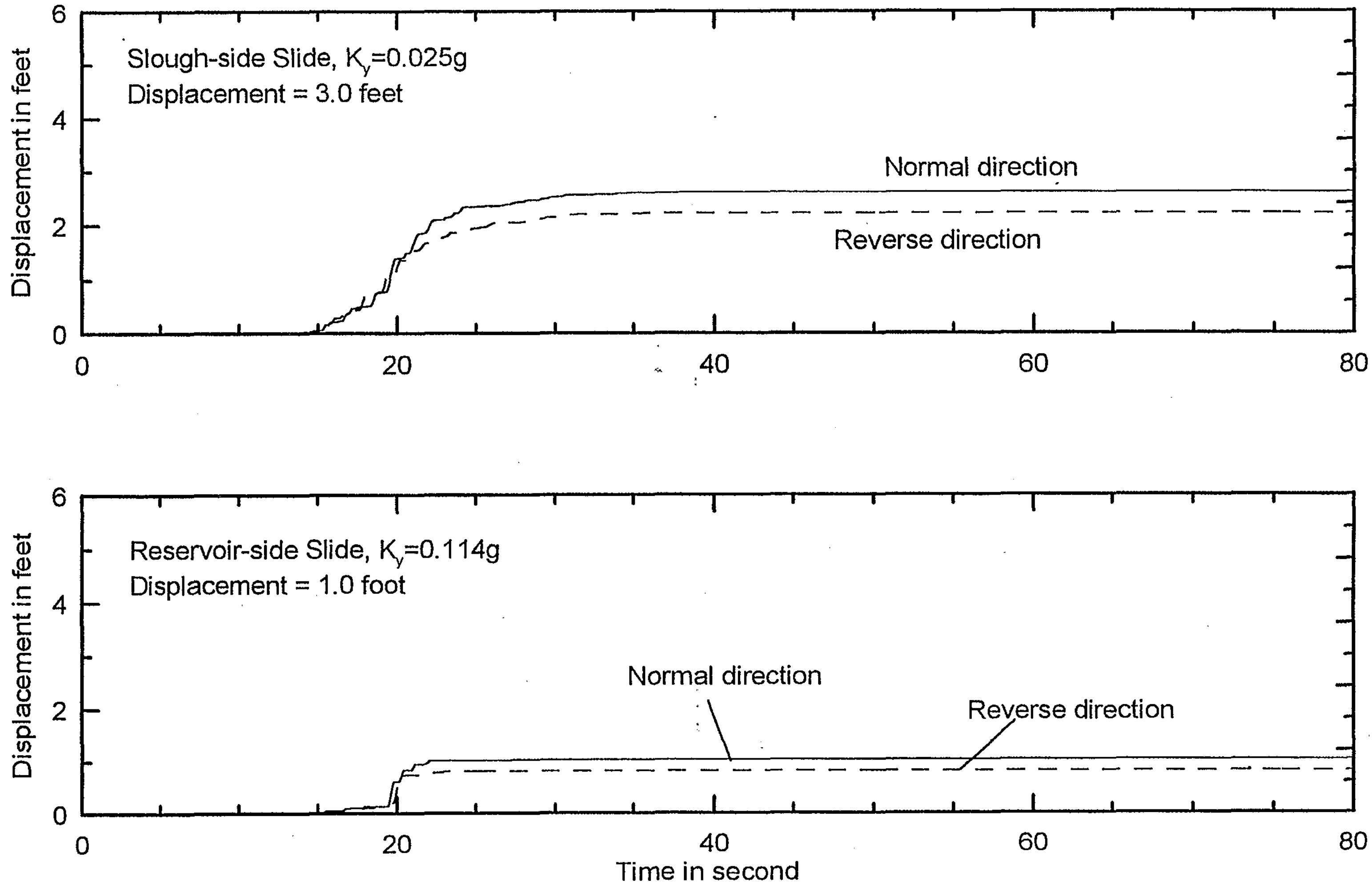




Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1987 Whittier Narrows Earthquake - Webb Tract Levee at Station 160+00	Figure A.12
URS Greiner Woodward Clyde			

C-063557

Webb Tract Levee at Station 160+00, Levee Deformations
1992 Landers Earthquake at St. 24577 (Fort Irwin), 0 Degree Component

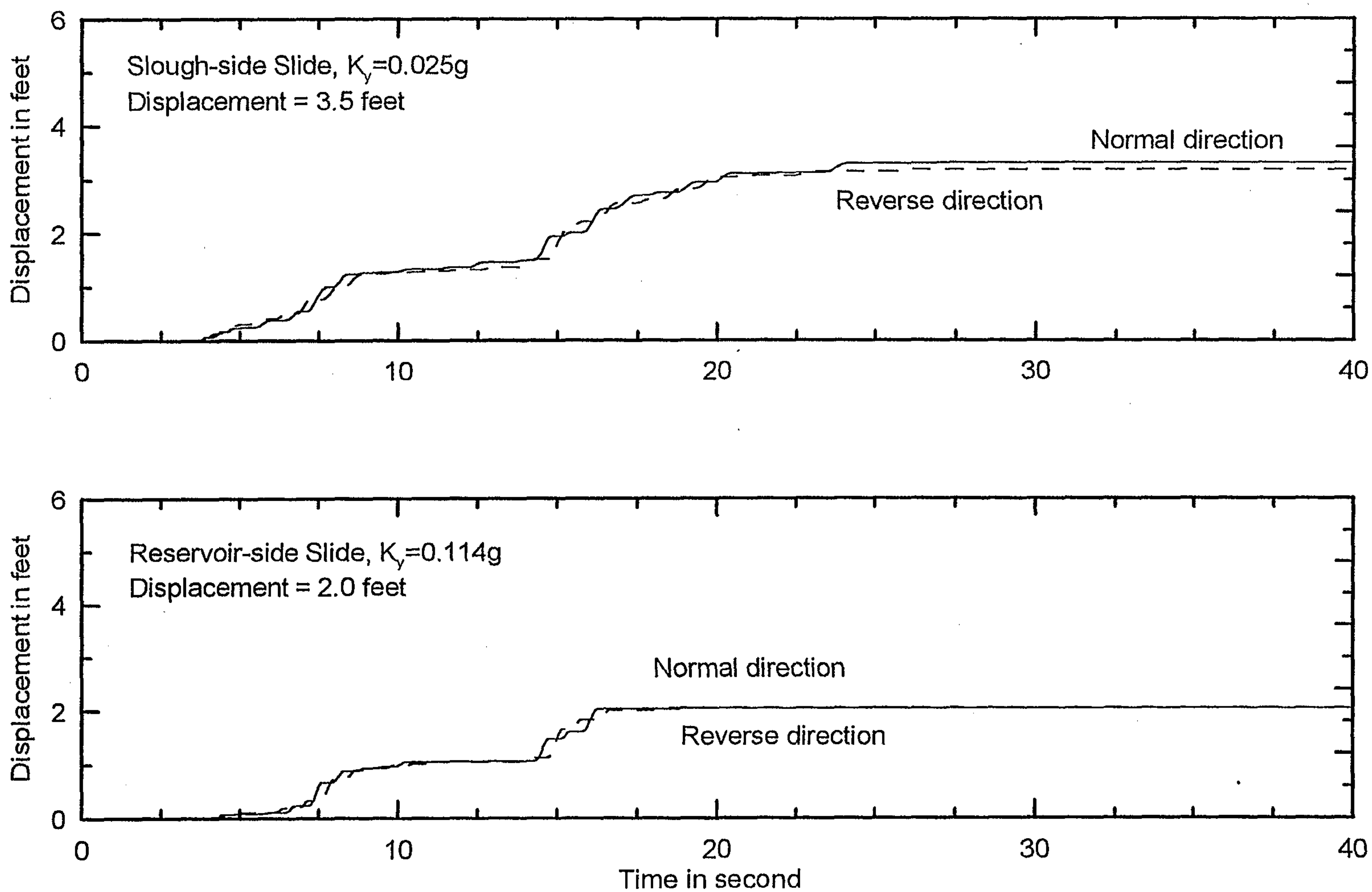


Project No. 410709903000	Delta Wetlands	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1992 Landers Earthquake - Webb Tract Levee at Station 160+00	Figure A.13
URS Greiner Woodward Clyde			

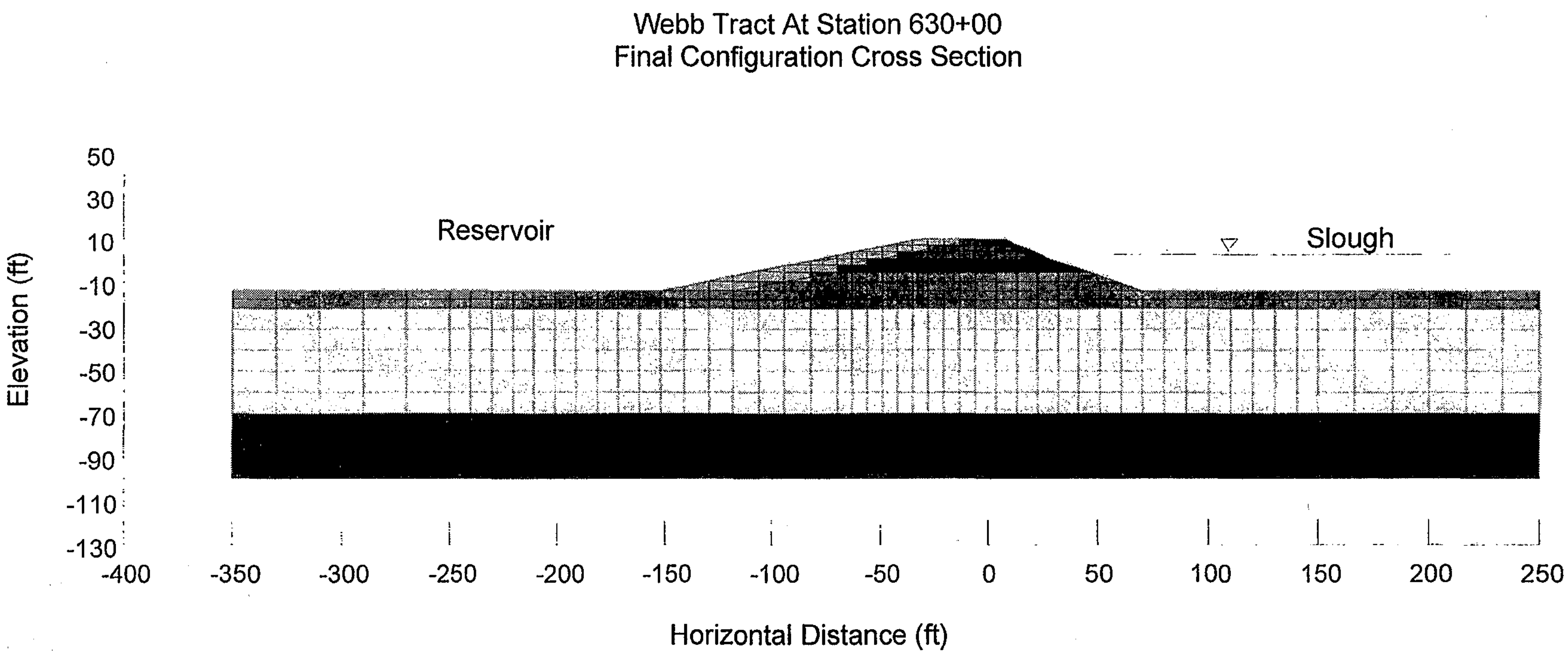
C-063557

Webb Tract Levee at Station 160+00, Levee Deformations

1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon),90 Degree Component



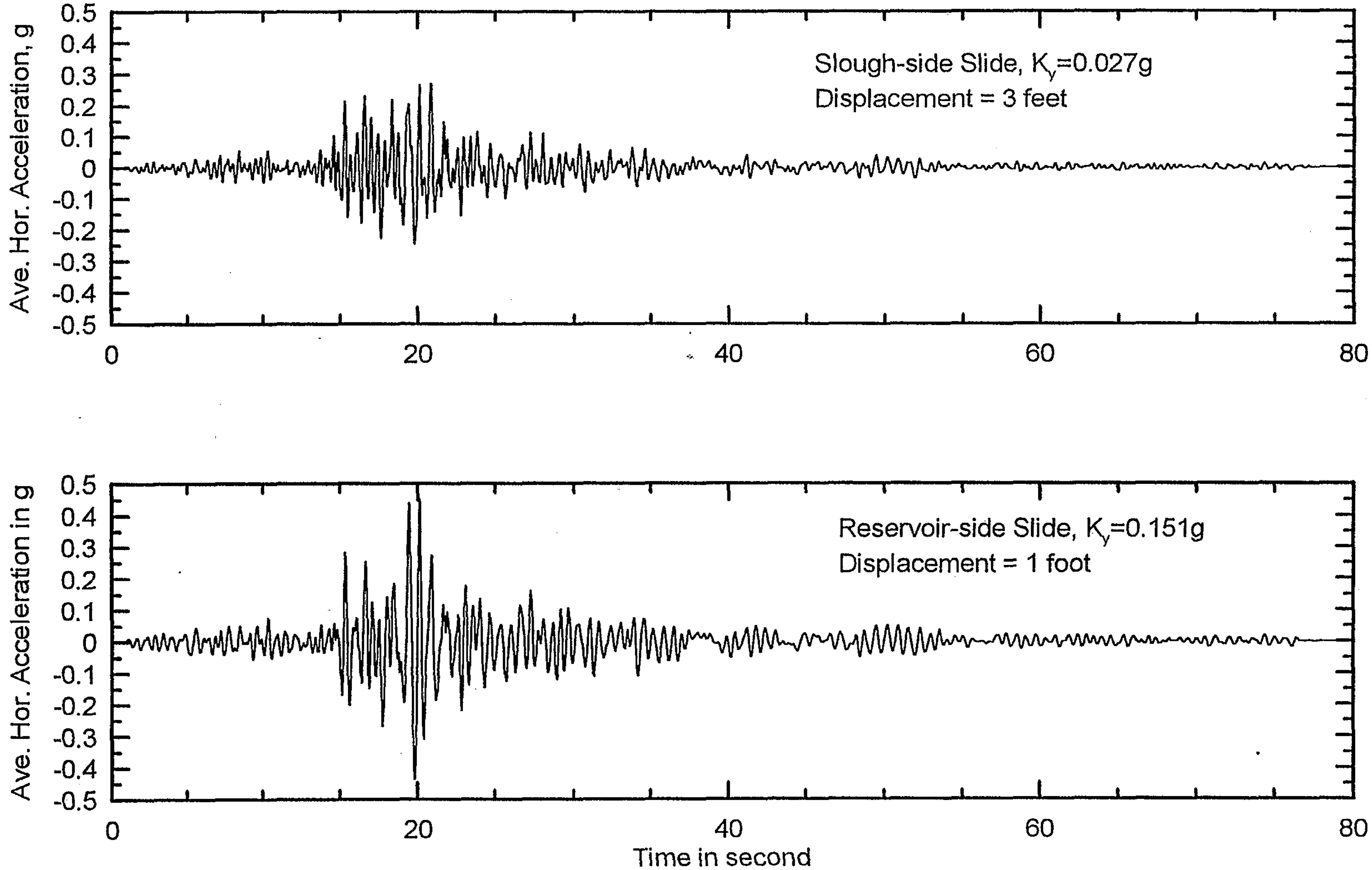
Project No. 410709903000	Delta Wetlands	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1987 Whittier Narrows Earthquake - Webb Tract Levee at Station 160+00	Figure A.14
URS Greiner Woodward Clyde			



Project No. 410709903000	Delta Wetlands	Finite Element Mesh for Webb Tract Levee at Station 630+00	Figure A.15
URS Greiner Woodward Clyde			

C-063560

Webb Tract Levee at Station 630+00, Dynamic Levee Responses
1992 Landers Earthquake at St. 24577 (Fort Irwin), 0 Degree Component



Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1992 Landers Earthquake - Webb Tract Levee at St. 630+00	Figure A.16
URS Greiner Woodward Clyde			

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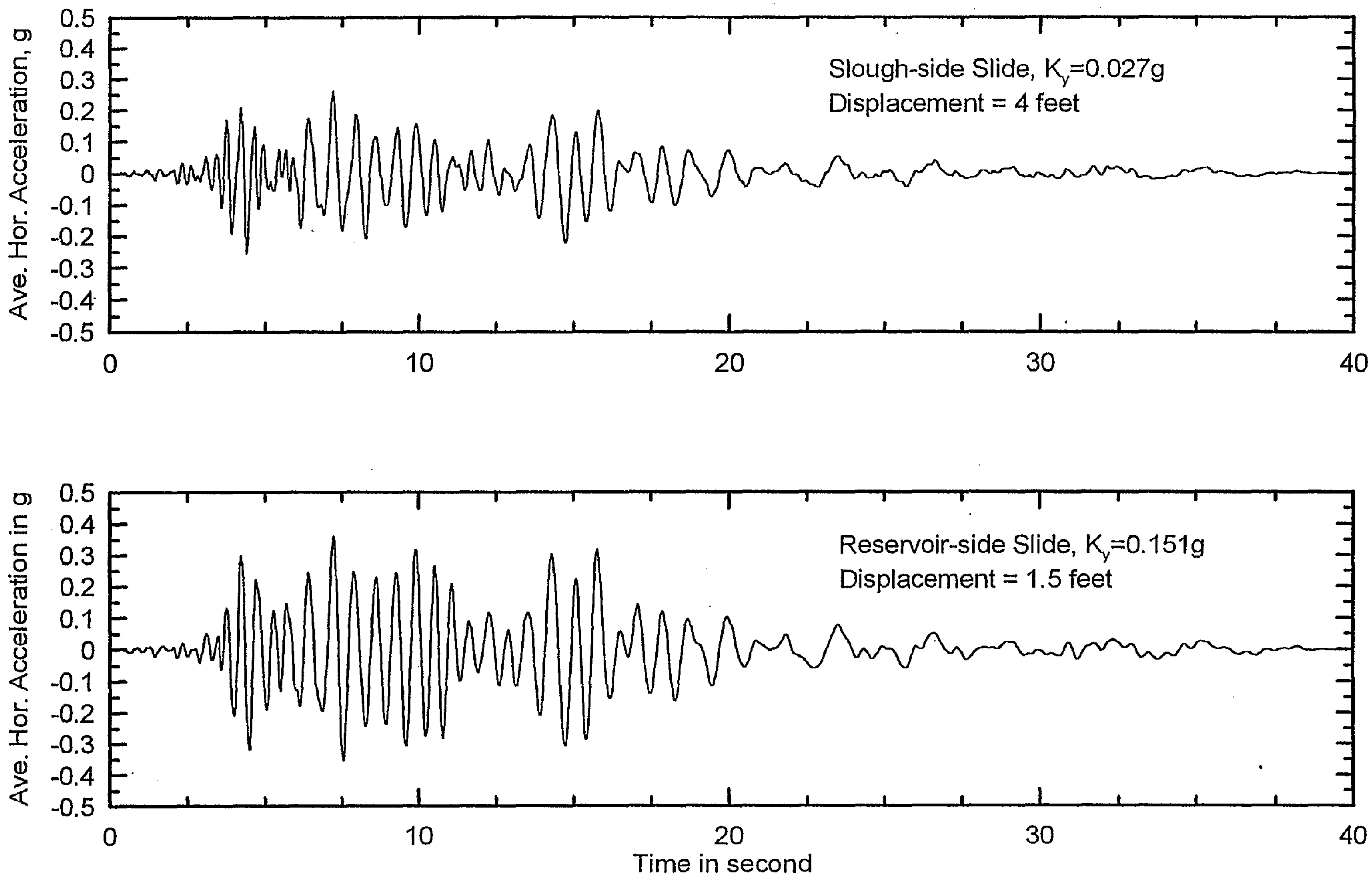
Webb Tract Levee at Station 630+00, Dynamic Levee Responses
1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon), 90 Degree Component



Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1987 Whittier Narrows Earthquake - Webb Tract Levee at Station 630+00	Figure A.17
URS Greiner Woodward Clyde			

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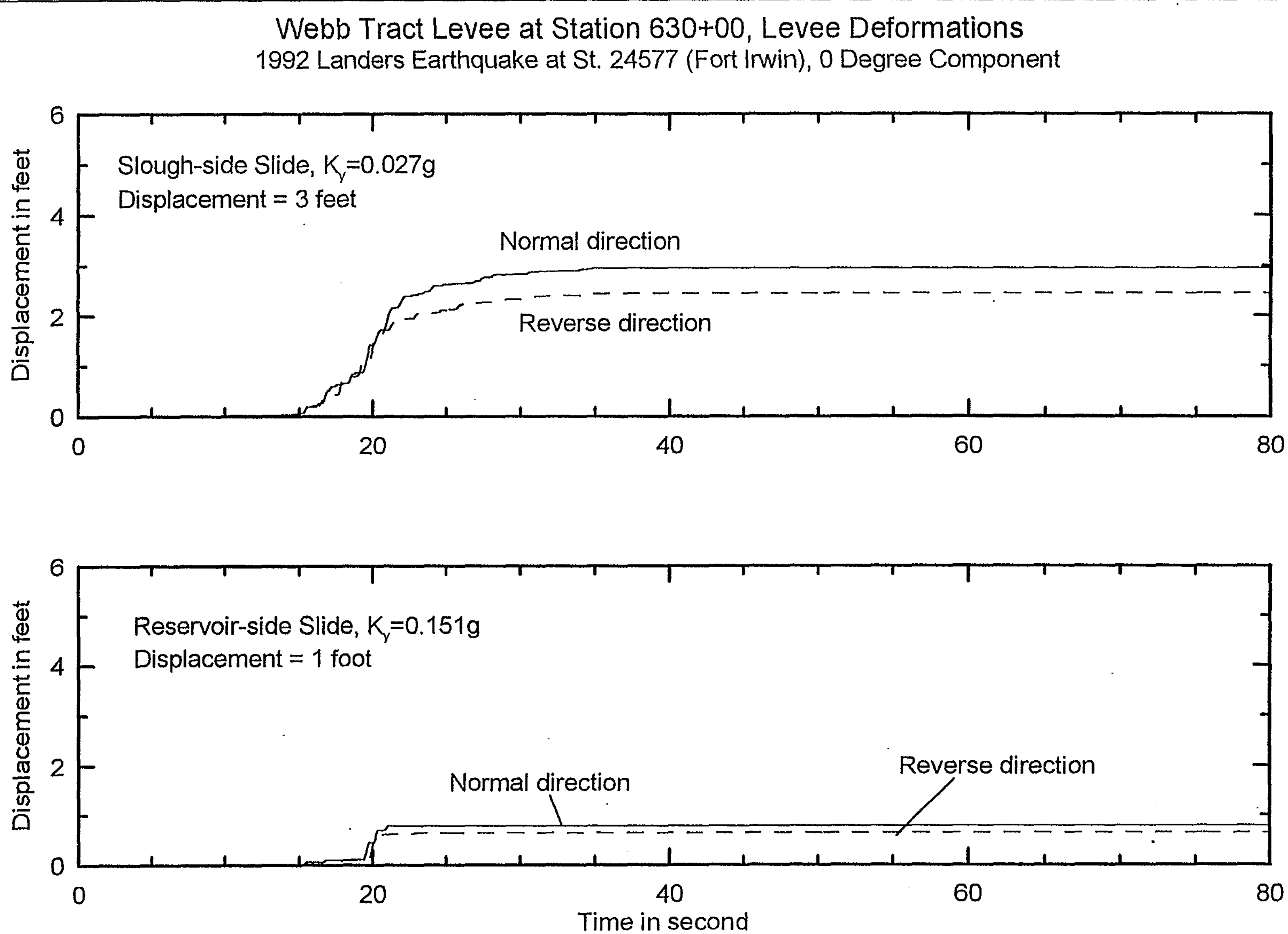
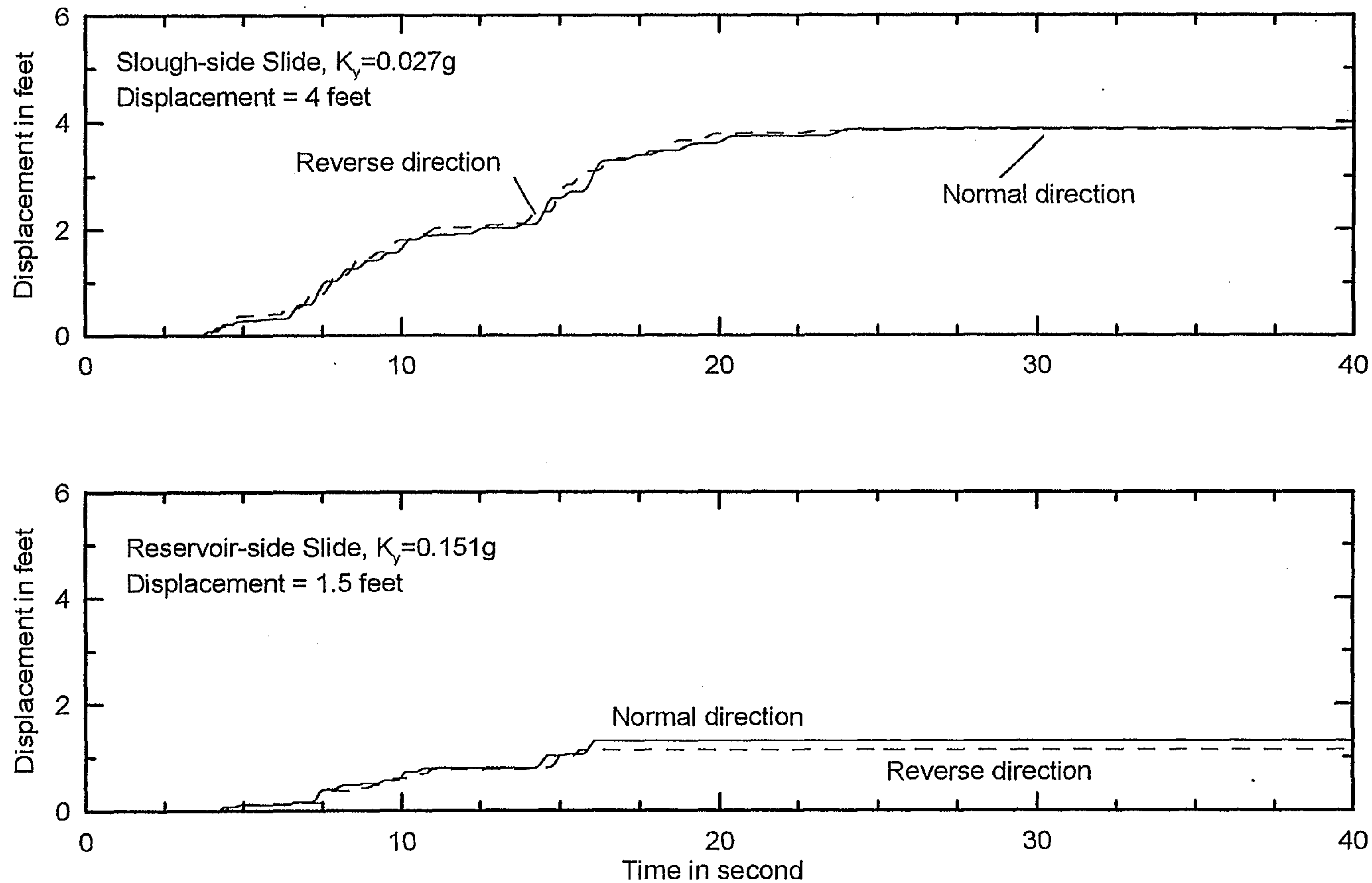


Project No. 410709903000	Delta Wetlands	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1992 Landers Earthquake - Webb Tract Levee at Station 630+00	Figure A.18
URS Greiner Woodward Clyde			

Webb Tract Levee at Station 630+00, Levee Deformations

1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon), 90 Degree Component



Project No. 410709903000	Delta Wetlands	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1987 Whittier Narrows Earthquake – Webb Tract Levee at Station 630+00	Figure A.19
URS Greiner Woodward Clyde			

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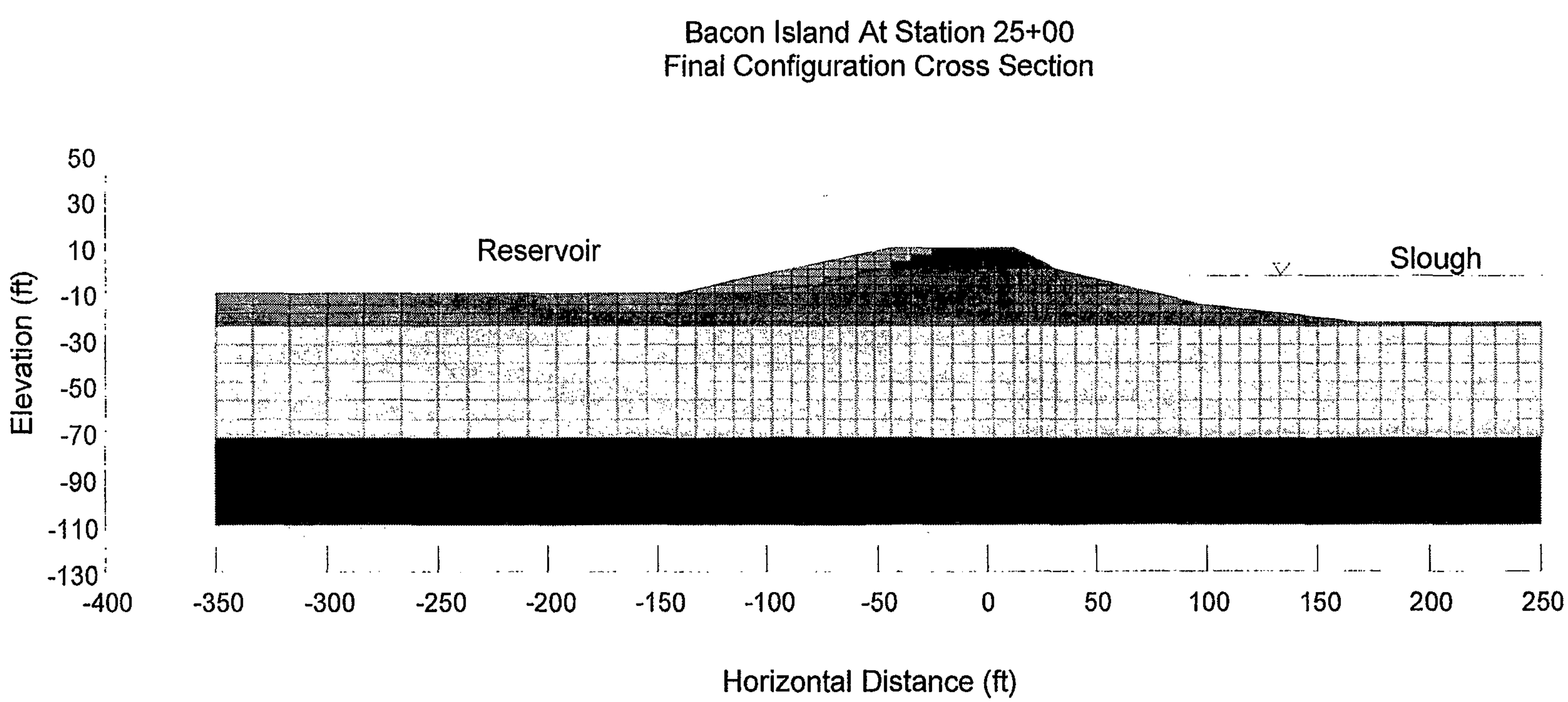

Project No. 410709903000	Delta Wetlands	Finite Element Mesh for Bacon Island Levee at Station 25+00	Figure A.20
URS Greiner Woodward Clyde			

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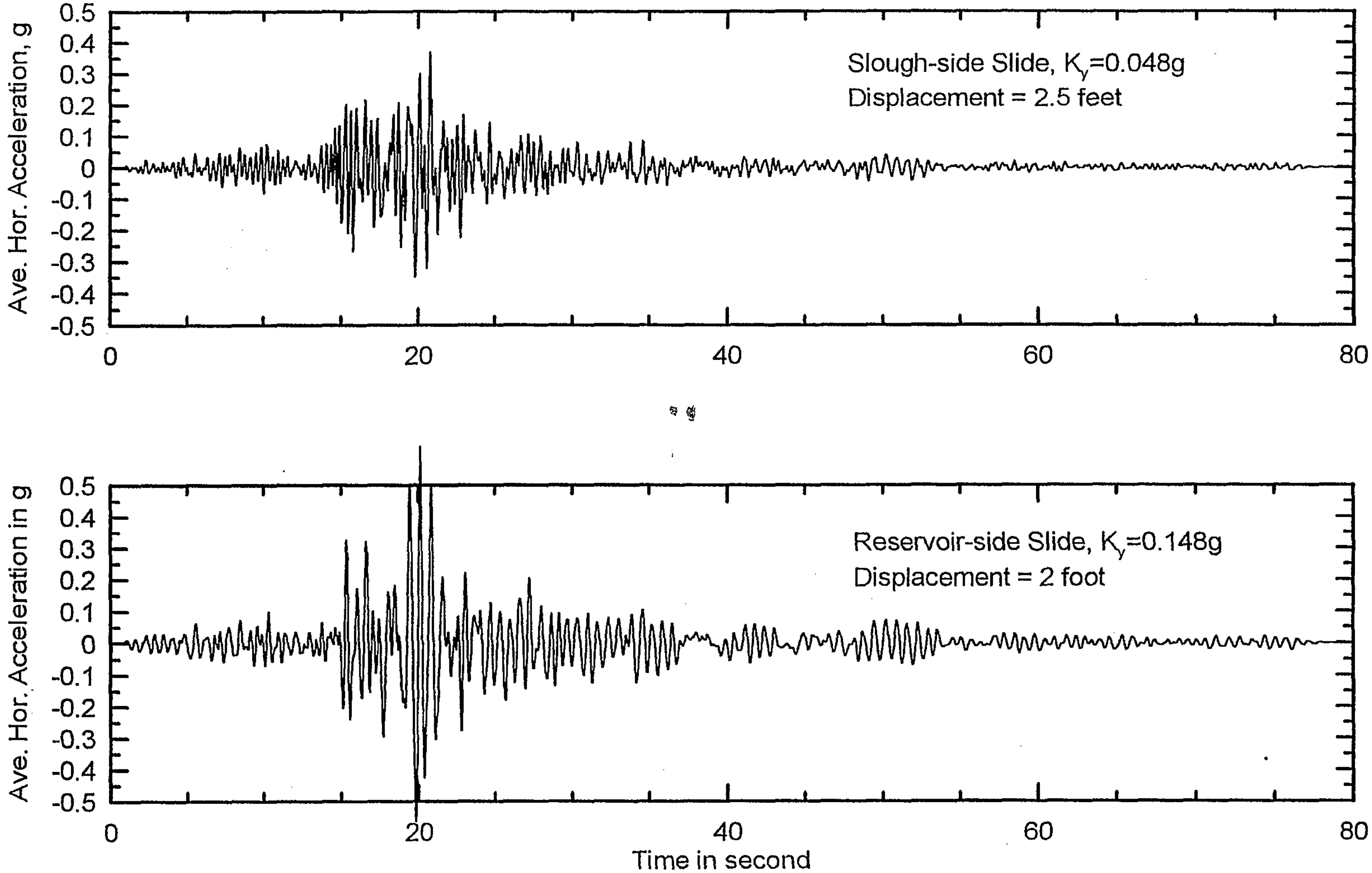
Bacon Island Levee at Station 25+00, Dynamic Levee Responses
1992 Landers Earthquake at St. 24577 (Fort Irwin), 0 Degree Component



Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1992 Landers Earthquake - Bacon Island Levee at St. 25+00	Figure A.21
URS Greiner Woodward Clyde			

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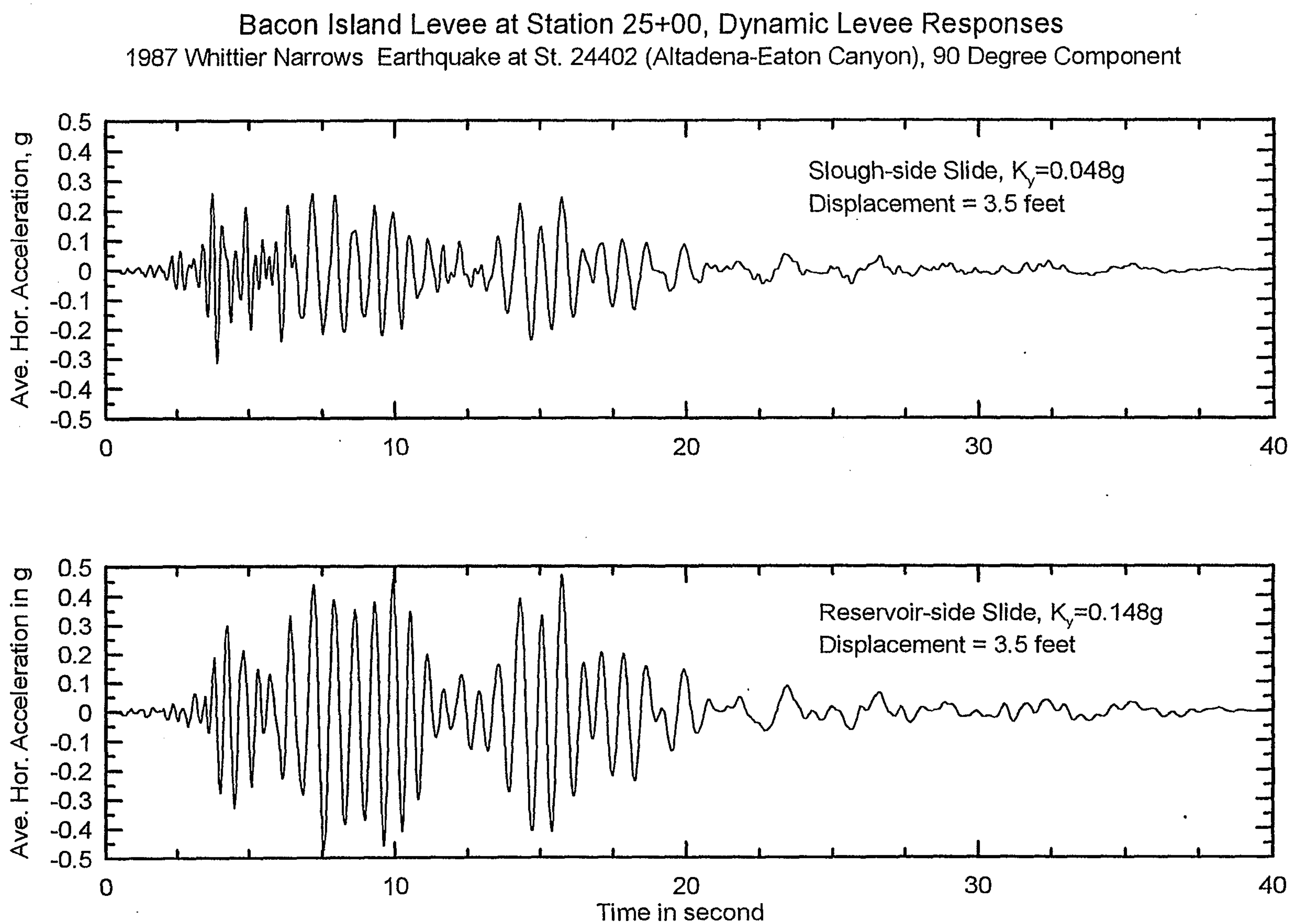

Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1987 Whittier Narrows Earthquake - Bacon Island Levee at Station 25+00	Figure A.22
URS Greiner Woodward Clyde			

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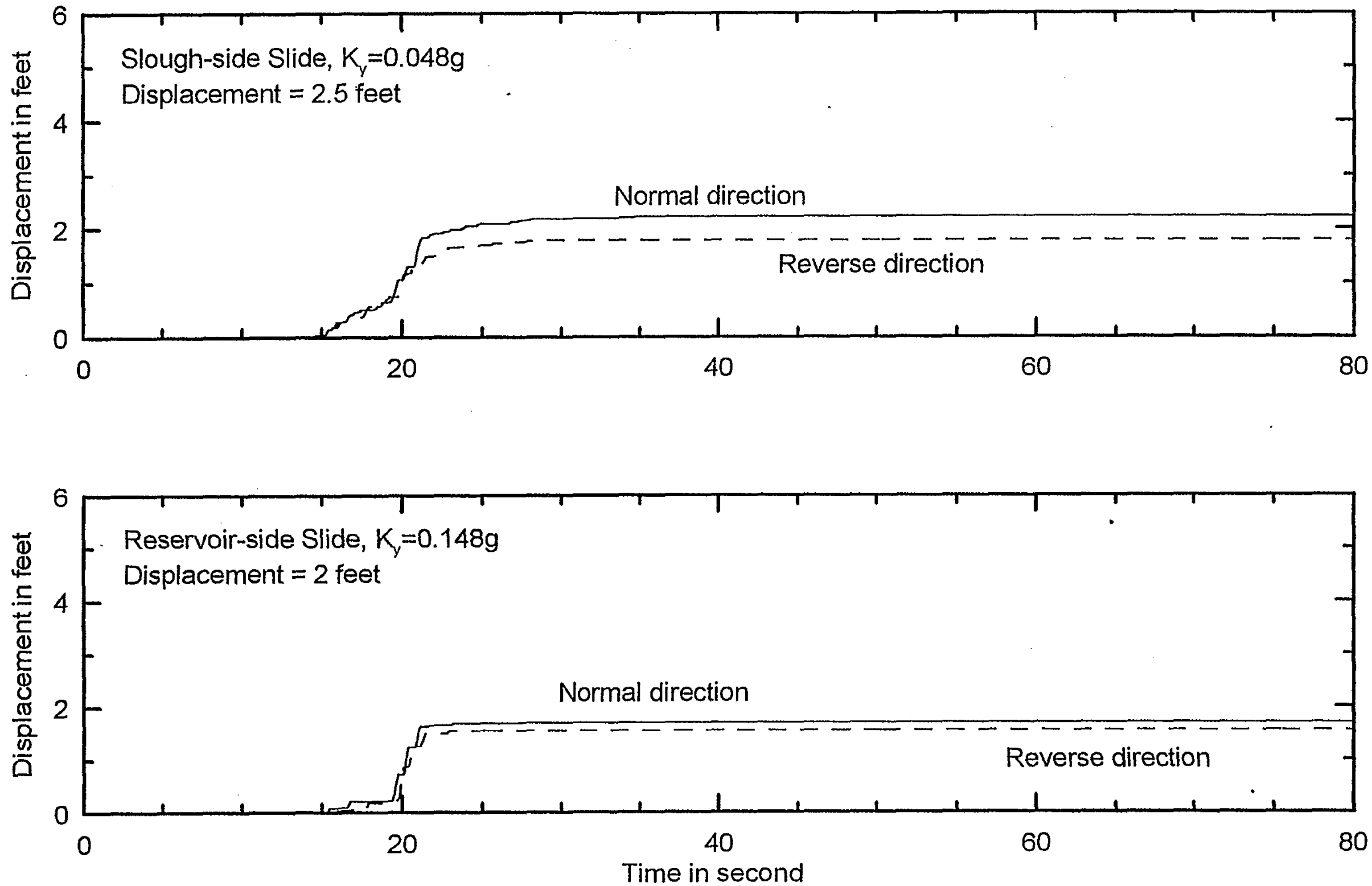
Bacon Island Levee at St. 25+00, Levee Deformations

1992 Landers Earthquake at St. 24577 (Fort Irwin), 0 Degree Component



Project No. 410709903000	Delta Wetlands	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1992 Landers Earthquake - Bacon Island Levee at Station 25+00	Figure A.23
URS Greiner Woodward Clyde			

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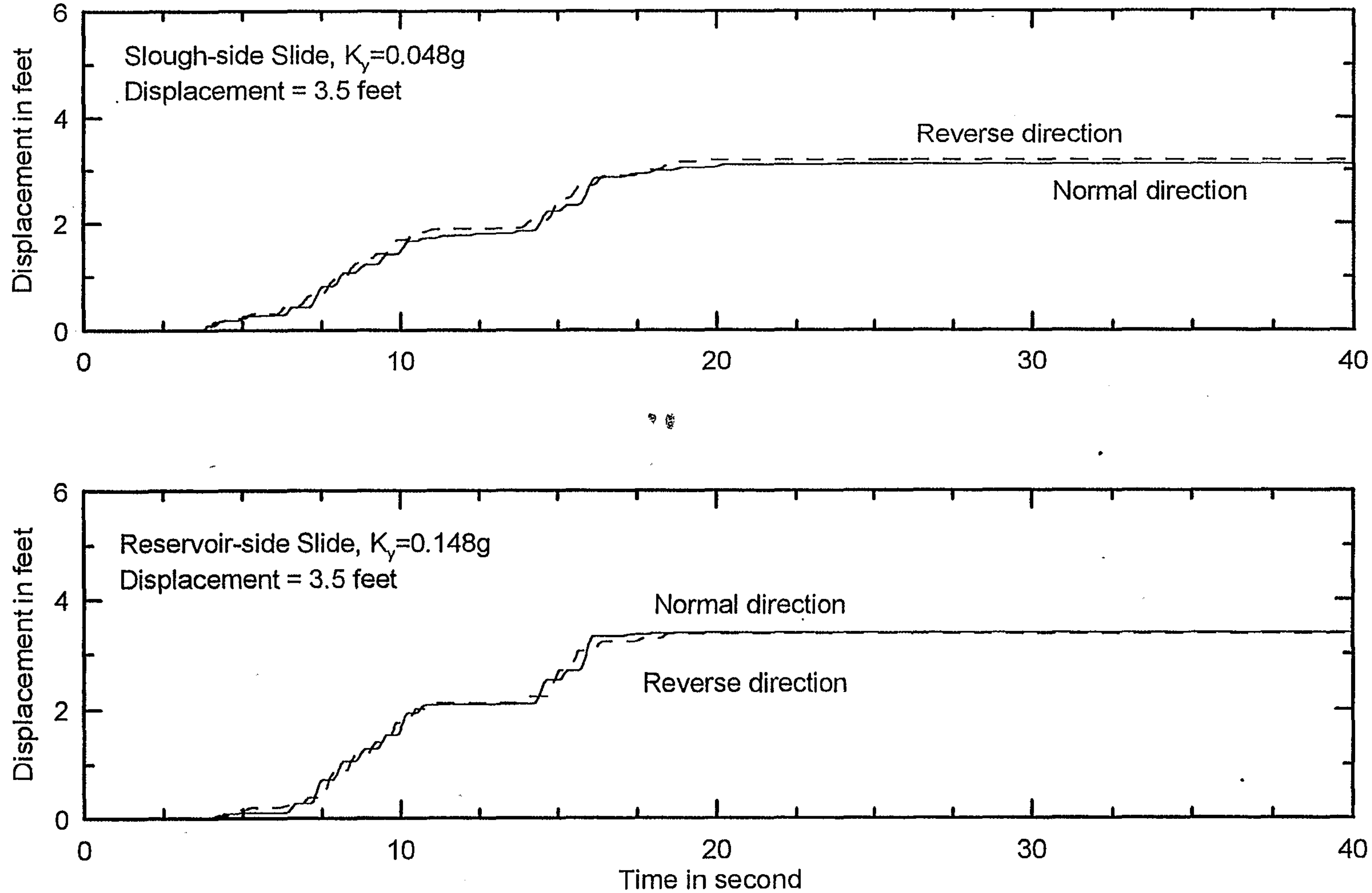
Bacon Island Levee at St. 25+00, Levee Deformations

1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon), 90 Degree Component



Project No. 410709903000	**Delta Wetlands**	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1987 Whittier Narrows Earthquake - Bacon Island Levee at Station 25+00	Figure A.24
URS Greiner Woodward Clyde			

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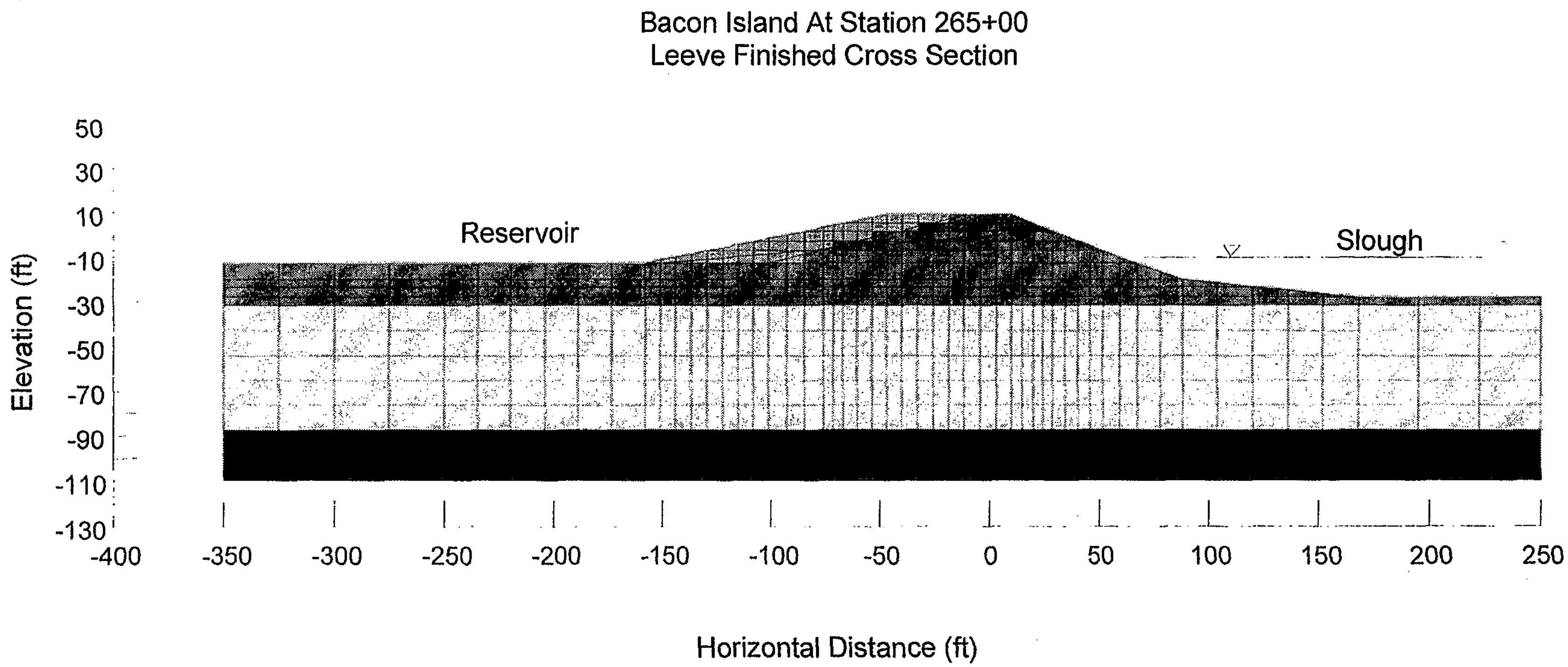
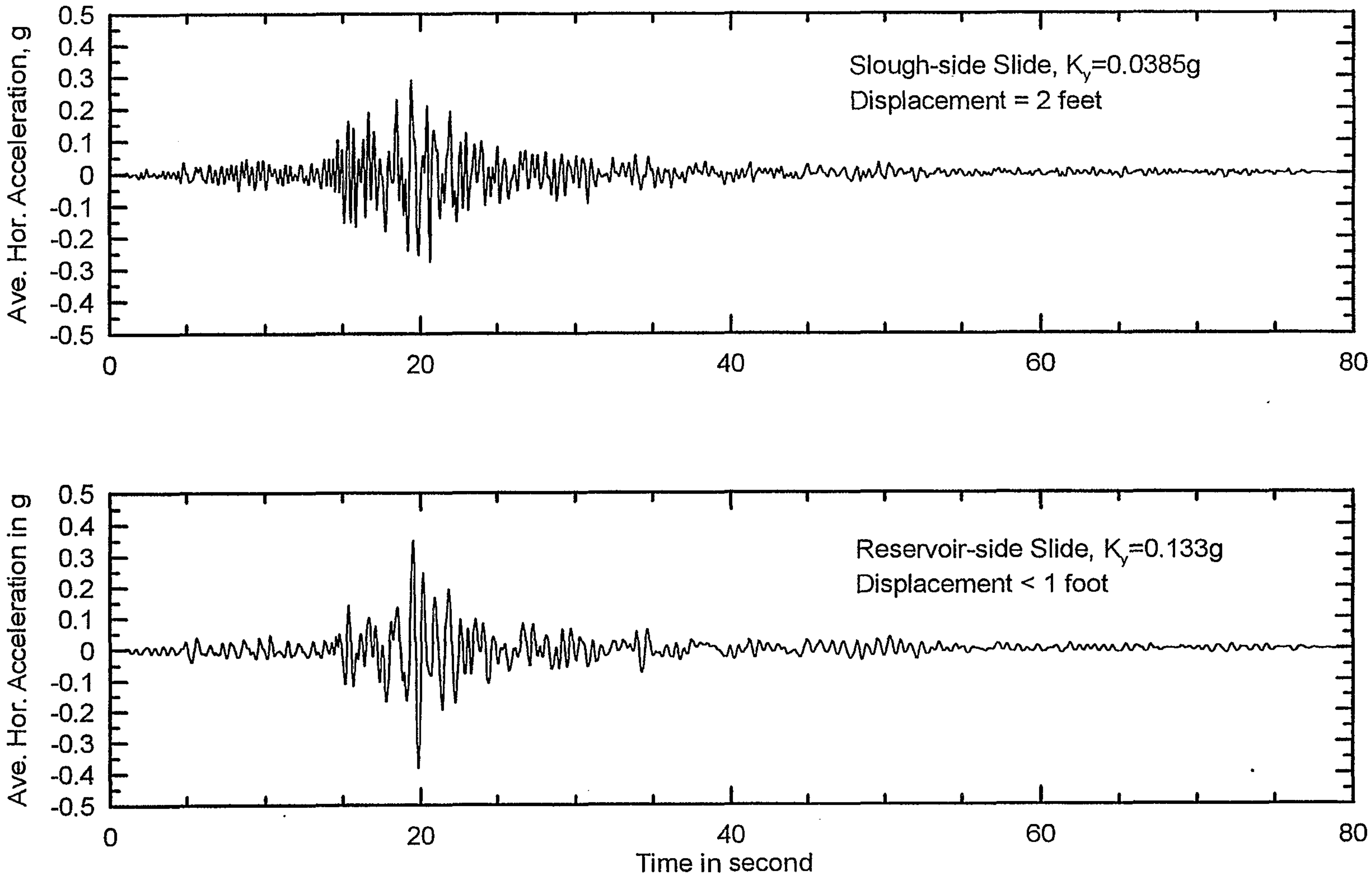


Project No. 410709903000	Delta Wetlands	Finite Element Mesh for Bacon Island Levee at Station 265+00	Figure A.25
URS Greiner Woodward Clyde			

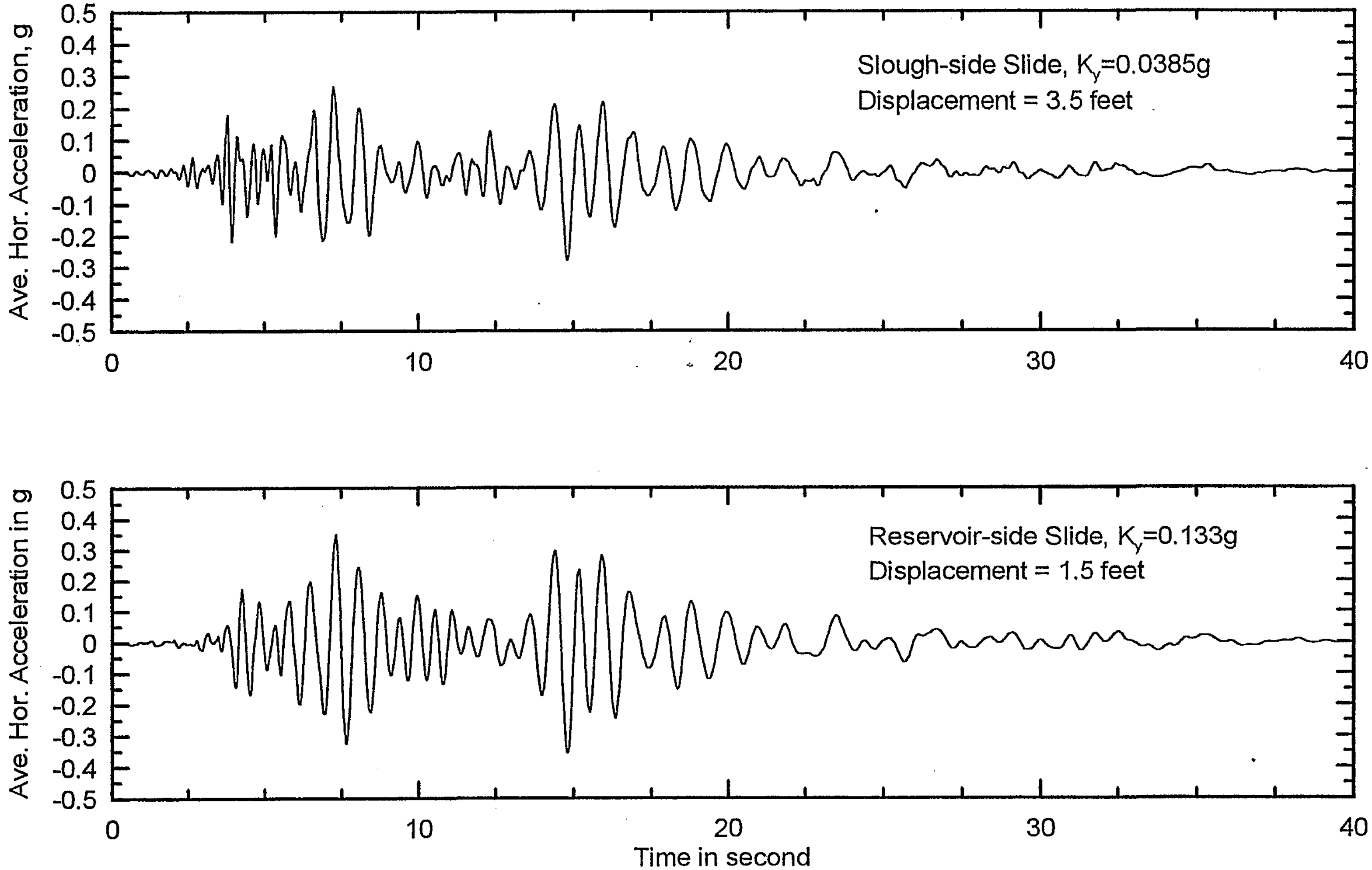
Bacon Island Levee at Station 265+00, Dynamic Levee Responses
1992 Landers Earthquake at St. 24577 (Fort Irwin), 0 Degree Component



Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1992 Landers Earthquake - Bacon Island Levee at St. 265+00	Figure A.26
URS Greiner Woodward Clyde			

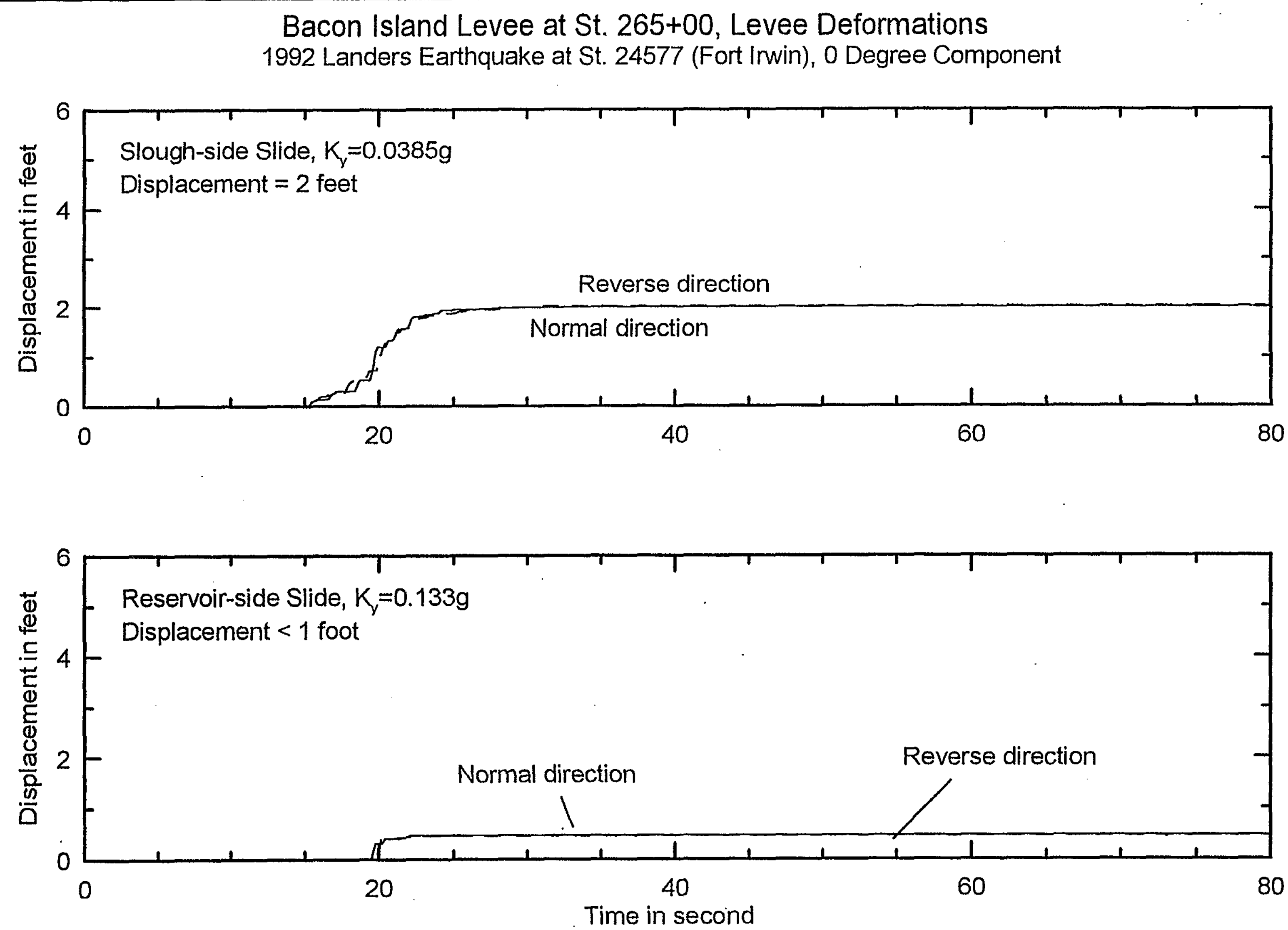
Bacon Island Levee at Station 265+00, Dynamic Levee Responses

1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon), 90 Degree Component



Project No. 410709903000	Delta Wetlands	Average Horizontal Acceleration Time Histories Acting on Critical Slide Masses for 1987 Whittier Narrows Earthquake - Bacon Island Levee at Station 265+00	Figure A.27
URS Greiner Woodward Clyde			

C-063571



Project No. 410709903000	Delta Wetlands	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1992 Landers Earthquake - Bacon Island Levee at Station 265+00	Figure A.28
URS Greiner Woodward Clyde			

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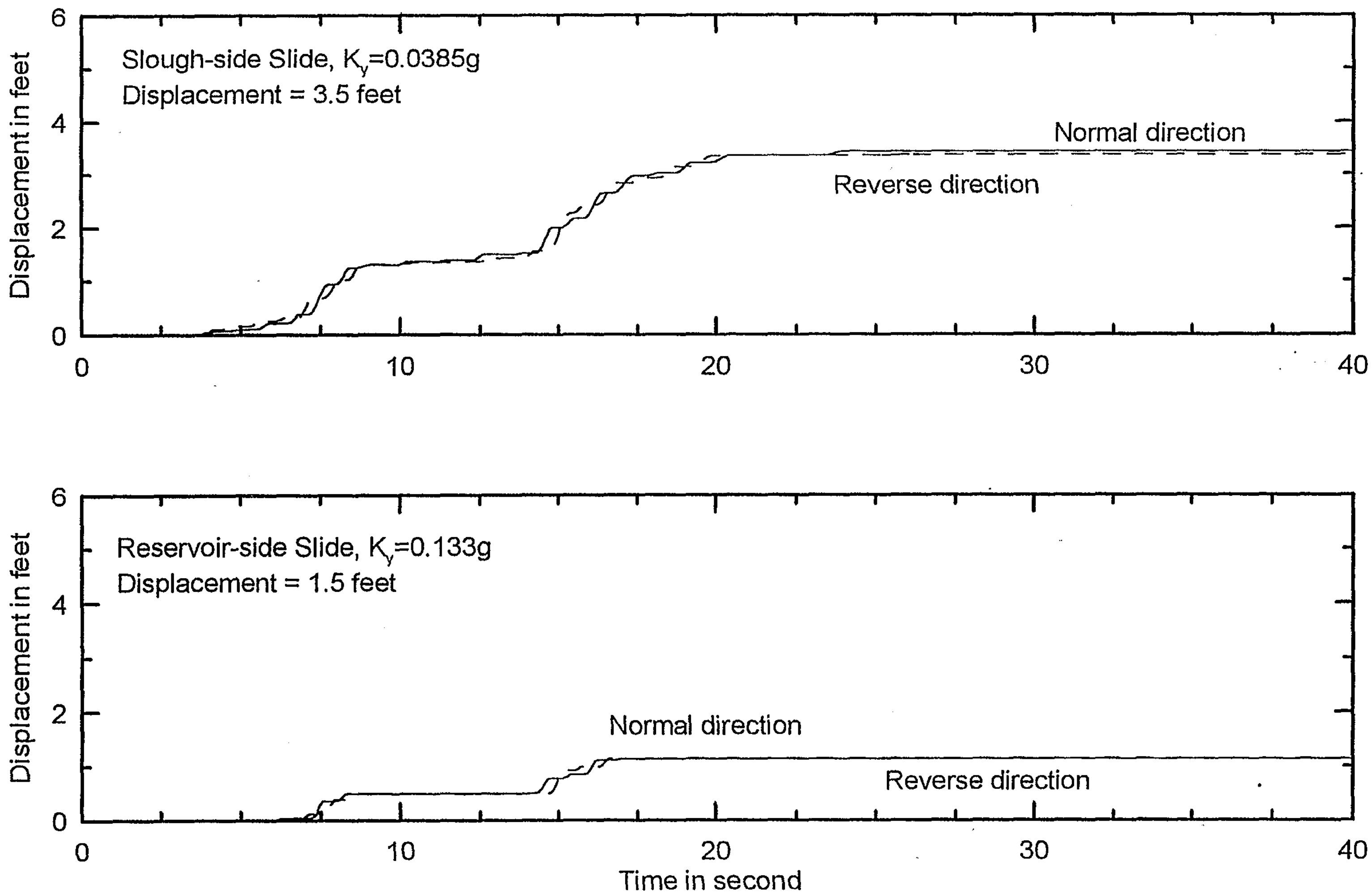
Bacon Island Levee at St. 265+00, Levee Deformations

1987 Whittier Narrows Earthquake at St. 24402 (Altadena-Eaton Canyon), 90 Degree Component



Project No. 410709903000	Delta Wetlands	Permanent Slope Deformation Time Histories of Critical Slide Masses for 1987 Whittier Narrows Earthquake - Bacon Island Levee at Station 265+00	Figure A.29
URS Greiner Woodward Clyde			

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Appendix B
Glossary

Anisotropy: The characteristic of having a physical property that varies with direction. For example, it is common for natural river deposits to be anisotropic, as their horizontal hydraulic conductivity can be several to many times higher than their conductivity in the vertical direction.

ASDSO (Slope Stability) Criteria: Set of recommended minimum slope stability Factors of Safety (see definition of Factor of Safety below) recommended for design of embankment dams developed by the ASDSO (Association of State Dam Safety Officials), established in 1983 and based in Lexington, Kentucky.

Cohesionless Soil: Soils like sands and gravel whose grains tend not that remain "stuck" together when free water has been drained. Cohesionless soils tend to let water drain easily. Opposite to this type of soil behavior is that of cohesive soils such as clay and silt. **Cohesive soils** are commonly referred to as fine-grained soils. Although not strictly a cohesive soil, peat is considered in analyses with engineering properties close to those of cohesive soil.

CPT (Cone Penetration Testing): CPT refers to a procedure in which a probe of conical shape is pushed into the soil at a constant rate while the tip and lateral resistances of the probe are measured electronically at regular depth intervals.

Damping Curve: Empirical relationship between damping and shear strain used to model the increase in damping value in cyclic loading.

Deterministic Ground Motions - earthquake ground motions associated with a specific seismic event occurring on a seismic source.

Elastic Half Space Model: A numerical model used to simulate a semi-infinite body of elastic material.

Exit Gradients: The hydraulic gradient that occurs at or just below the ground surface. Excessively high exit gradients can cause a "quick" or "boiling" effect and piping, under which materials can lose strength and be carried away.

Factor of Safety (Slope Stability): The Factor of Safety (FS) is a calculated number representing the degree of safety of a slope against instability. The FS is expressed mathematically as the ratio of stabilizing effects (forces or moments) and destabilizing effects acting on a potentially unstable soil mass in a slope. When the FS is greater than one, the soil mass in the slope is, in theory, stable; when FS is lower than 1, the slope is, in theory, unstable. For a given slope geometry and soil conditions, a calculated FS is associated with a unique slope failure configuration. The most critical failure configuration is associated with the minimum FS calculated in a slope stability analysis. Several agencies (such as ASDSO and USACE) have developed criteria that provide different design FS's stipulated for various slope conditions, e.g. under long-term loading, shortly after construction, etc. These design FS's are typically above one and are minimum values to be achieved for the slope to be considered stable.

Gravity Flow Relief Wells: Wells that provide a means of water release for subsurface sources (typically those under conditions of excess pressure) by providing a free-flowing outlet source. These types of wells do not draw water with pumps, but simply use an outlet source that relies on gravity flow, such as a low-lying outlet or drainage ditch.

Hydraulic Conductivity: A measure of the capacity of a porous medium to transmit water, often expressed in centimeters per second. The hydraulic conductivity is equal to the rate of flow of water through a cross section of one unit area under a unit hydraulic gradient.

Hydraulic Gradient: The rate of change in total hydraulic head per unit of distance of flow in a given direction.

Ky: Pseudo-static horizontal acceleration that will give a calculated factor of safety in slope stability analyses of 1.0; yield acceleration.

Maximum Credible Earthquake (MCE): The maximum earthquake that appears capable of occurring under the presently known tectonic framework. It is a rational and believable event that is in accord with all known geologic and seismologic facts. In determining the maximum credible earthquake, little regard is given to its probability of occurrence, except that its likelihood of occurring is great enough to be of concern. (from CDMG Note Number 43, February 1975). The Uniform Building Code (ICBO, 1991) defines the MCE for seismic-isolated structures as the maximum level of earthquake ground shaking which may ever be expected at the building site within the known geologic framework. For this case, the UBC says that the MCE may be taken as the ground motion that has a 10 percent probability of being exceeded in 250 years.

Most Critical (Water Level) Case: Water level on either side of the levee for the case with stored water and during seismic event. On the channel side, the water level changes daily with the tide and seasonally; on the island side, the (ground) water level also varies. For seismic analyses, these levels are assumed at their expected, mean values. Because it is highly unlikely that the design seismic event that occurs at the same time extreme water levels take place, mean water levels are considered for the seismic case.

Peak Ground Acceleration (PGA): The maximum value of acceleration recorded on a seismograph during an earthquake event.

Permeability: A measure of the capacity of a porous medium to transmit a fluid. See Hydraulic Conductivity.

Piping: The removal of fine soil particles from the soil mass by high hydraulic gradients. For example, excessively high exit hydraulic gradients at the surface may cause upward transport of soil, resulting in sand boils.

Poisson Earthquake Model: A model of earthquake recurrence in which the inter-occurrence time is random and does not depend on the time of the last event.

Pseudo-static Analysis: Seismic slope stability analyses using a static force that is equivalent to the horizontal acceleration experienced during a seismic event.

Pumped Extraction Wells: Wells that draw water from subsurface sources by powered mechanical or hydraulic pumps.

Relief Wells: Wells that passively relieve elevated hydrostatic pressures in an aquifer by allowing flow to the surface.

Representatively Critical (Water Level) Case: Water level on either side of the levee for the existing conditions (case with no stored water). On the channel side, the water level changes daily with the tide and seasonally; on the island side the (ground) water level also varies. For engineering analyses, these levels are assumed at their expected, representative maximum and representative minimum values. Less likely to occur extreme values are not considered in the existing conditions case.

Return Period: The average time between events. Typically, events are defined as the occurrence of an earthquake exceeding a specified magnitude or the occurrence of a ground motion greater than a specified level.

Seepage Flux: The rate of flow of water across a given line or surface, typically expressed in gallons per minute (gpm) or cubic feet per second (cfs). For example, in the finite element model SEEP, the average seepage flux through a levee can be estimated using a cross-sectional model and expressed in gpm per foot of levee length.

Shear Modulus Reduction Curve: Empirical relationship between the ratio of shear modulus and its maximum value and the shear strain used to model the degradation in shear modulus in cyclic loading.

SPT (Soil Penetration Testing): SPT refers to the procedure to determine the soil penetration resistance. In general terms, the penetration resistance is measured by counting the number of blows necessary to drive a soil sampler (steel tube) a specified distance using a hammer of a specified weight into the subsoil at the bottom of the borehole. In the standardized test, commonly referred to as the Standard Penetration Test, the hammer is 140 pounds and is dropped repeatedly 30 inches; the sampler is 1-3/8-inch I.D., 2-inch O.D., and is driven 18 inches into the soil. The penetration resistance is computed by adding the blow counts recorded for last two 6-inch increments of driving length.

Transmissivity: The transmission capability of the entire thickness of an aquifer, often expressed in gallons per day per foot of aquifer thickness. The transmissivity can be determined by multiplying the hydraulic conductivity by the aquifer thickness.

Visco-elastic Material: A material that behaves elastically and absorbs energy (damping).

Appendix C

State Water Resources Control Board Letter Dated November 1998

State Water Resources Control Board

John P. Caffrey, Chairman

Peter M. Rooney
Secretary for Environmental Protection

Executive Office
901 P Street • Sacramento, California 95814 • (916) 657-0941 • FAX (916) 657-0932
Mailing Address: P.O. Box 100 • Sacramento, California 95812-0100
Internet Address: http://www.swrcb.ca.gov

Pete Wilson
Governor

NOV 2 5 1998

Anne Schneider, Esq.
Ellison & Schneider
2015 H Street
Sacramento, CA 95814-3109

Dear Ms. Schneider:

DELTA WETLANDS PROJECT, WATER RIGHT APPLICATIONS 29061, 29062, 29063, 29066, 30267, 30268, 30269, AND 30270: THE NEXT STEPS

As you know, the State Water Resources Control Board (Board) has considered action on the subject applications at two recent executive sessions. The Board has directed me to contact you and seek Delta Wetlands' input before deciding upon a further course of action. That is the purpose of this letter. Among the factors that need to be considered are the following:

- If the project is to be wholly or partially approved the Environmental Impact Report (EIR) will need to be completed. Additional information must be added before the final EIR can be prepared, and this may require recirculation of the document. Because of the additional expense involved, we have not directed Jones & Stokes (JSA) or our staff to proceed with completion as yet. Your response to this inquiry will help determine the course of action.

- You requested inclusion of the new Department of Fish and Game (DFG) Biological Opinion (BO) in the hearing record. Several parties have requested further hearing to allow the cross-examination of DW and DFG witnesses and to submit rebuttal evidence. If the project proceeds, that request will be granted, and the BO will be a subject of hearing.

- On October 21, 1998, you requested certification of the project under section 401 of the federal Clean Water Act (CWA), a necessary prerequisite to issuance of a CWA section 404 permit by the Corps of Engineers. The remaining time within which the Board can act upon that request is very limited, and the Board cannot approve a certification request without a completed California Environmental Quality Act (CEQA) document. As you pointed out, your letter triggered a 60-day time frame for the SWRCB to act on the certification. The

EIR cannot be made final and be certified before the end of the 60-day period triggered by your letter. Therefore, I intend to deny 401 certification, without prejudice, pending completion of an appropriate CEQA document. I will send you a separate letter on that subject.

- Review of the existing hearing record reveals substantial remaining uncertainty regarding several significant issues. A summary of those issues is contained in Attachment A. The Board has made no determination as yet regarding these issues, and any apparent conclusions in Attachment A are preliminary. It is possible that further hearing days will be required in an attempt to better document the potential project effects.

- Review of the existing hearing record reveals no assurance that if the project is constructed and later is abandoned, the cost of mitigation of the project will not be transferred to the public through default. Because of the location of the proposed project and the potential for adverse effects if it is not actively operated, a financial surety may be necessary to ensure that the taxpayers will not bear the burden of mitigation and dismantling costs.

- The Board is aware that the hearing record is well over a year old. Further information developed by others (e.g., Cal-Fed) may be useful in analyzing the issues described above. The applicant, and potentially other entities, may be interested in presenting additional information which bears on project feasibility and methods of dealing with some of the potential adverse impacts.

In view of the foregoing, the Board seeks an expression of preference from the applicant as to alternative courses of action. The alternatives available to the Board are:

1. The Board would issue a decision based upon the existing record.[1]

2. Further hearing would be conducted solely on the new BO. The Board would then issue a decision.

3. Further hearing would be held on the BO and to obtain more evidence on the issues described above and discussed in Attachment A, prior to a Board decision.

[1] A further hearing regarding the new BO appears to be necessary before approving the project.

A decision after a further hearing could entail (1) approval of the project with appropriate conditions, (2) denial and cancellation of the permit applications and change petitions, or (3) denial of the requested applications and petitions without cancelling them and without prejudice to further efforts by the applicant to support the project.

The Board appreciates your efforts and requests your input before deciding on a course of action. Other parties also are welcome to provide input in response to this letter. Please contact me at (916) 657-0941 if you have procedural questions.

Sincerely,

Walt Pettit

Walt Pettit
Executive Director

Attachment

cc: Delta Wetlands Mailing List

bc: SWRCB Members
Walt Pettit, EXEC
Dale Claypoole, EXEC
William Attwater, OCC
Andy Sawyer, OCC
Barbara Leidigh, OCC
Harry Schueller, DWR
Jerry Johns, DWR
Jim Sutton, DWR
Dave Cornelius, DWR
Jim Canaday, DWR

BJLeidigh/mkschmidgall
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ATTACHMENT A

I. WATER RIGHT CONSIDERATIONS

DW has stated its intention to withdraw Applications 29061, 29063, 30267, and 30269 to divert water to Bouldin Island and to Holland Tract. DW has designated these two islands as wildlife habitat islands, to offset potential wildlife and wetland impacts of the reservoirs it plans for Webb Tract and Bacon Island. DW claims existing water rights to use water on the habitat islands. Accordingly, the following discussion does not pertain to Applications 29061, 29063, 30267, and 30269.

I. A. No Identified Buyers for the Water

DW[1] presented no evidence that it has any buyers for the project water. The hearing record indicates that DW is not likely to have buyers. DW estimated that the project water would cost buyers in the range of $200 to $300 per acre-foot plus conveyance charges. This estimate may be low. Mitigation and operational limitations in addition to those assumed by DW would be needed before the project could be constructed and operated. In addition, the estimate does not include conveyance charges. At the price DW estimated, agricultural water users are unlikely to buy water from the project. Further, CCWD and CUWA, the representatives of the municipal water purveyors who would be most able to pay the relatively high price that would be asked for the water, provided evidence that the water is likely to substantially increase their water treatment costs. They are looking for ways to get water with less dissolved organic material in it than they currently receive from the Delta. The DW water, and Delta water to which DW water has been added, would generally contain more dissolved organic material than current Delta supplies. Consequently, CCWD and CUWA stated that they will not buy the water and do not want to receive it through DWR or USBR facilities.

With no buyer for the water, there would be no beneficial use of the water. In the absence of a beneficial use of the water, there can be no water right. (Wat. Code § 1240.) Lack of a buyer alone is not a fatal defect, since a permit could be conditioned to require a buyer to be identified before the project is constructed or the reservoirs filled. But DW has failed to establish any likelihood that a buyer will be found.

I. B. Water Availability Considerations

Water likely would be available to the DW Project due to high flows during winter months for limited periods, even in some dry years. DW based its estimate of water availability on a hydrological model referred to as DeltaSOS (DW 63.), which was developed by the EIR/EIS consultant. The model runs take into account the Final

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[1] The following abbreviations are used in this Attachment: DW means Delta Wetlands; CCWD means Contra Costa Water District; EBMUD means East Bay Municipal Utility District; DFG means Department of Fish and Game; PG&E means Pacific Gas and Electric Co.; DWR means Department of Water Resources; USBR means United States Bureau of Reclamation; SWRCB means State Water Resources Control Board; CUWA means California Urban Water Agencies; SWP means State Water Project; CVP means Central Valley Project; CALTRANS means California Department of Transportation; and CDWA means Central Delta Water Agency.

Operations Criteria for the biological opinions prepared under the federal Endangered Species Act. The DW analysis suggests that there will be water available for appropriation not only during and after winter storms when high flows are present in the Delta, but also occasionally in the drier months. The analysis shows no water available during April and May because of fish protections, but in all other months the analysis shows that some water will be available in at least some years. The analysis predicts that the largest average amounts of water would be available in October through February, with smaller average amounts available in September and March. (DW 8, table 13.)

The DeltaSOS model runs do not assume that DW would be required to avoid impacts on the diversions of water by CCWD under its senior water rights. DW argued that it did not need to avoid impacts on CCWD's diversions because the relevant constraint on CCWD's diversions is imposed under the Endangered Species Act, not under CCWD's water rights. CCWD's authority to divert water under its own water rights is dependent on the location of the two parts per thousand salinity line (X2) in the Delta.[2] DW argued that this is not a restriction on CCWD's senior water rights, and that DW should not be required to defer to it. CCWD requested that an X2 condition be placed in any permits issued to DW in order to protect CCWD's diversions. (CCWD 3, pp. 12-13.) CCWD's water right permits require CCWD to meet the Endangered Species Act requirements. Although the Endangered Species Act requirements for CCWD could change, CCWD's violation of the requirements that are in effect would simultaneously be a violation of its water right permits. If DW were issued permits, we expect the permits would be conditioned to prevent diversions by DW during periods when CCWD is unable to divert because of the location of X2 in the Delta.

Further, all diversions by the DW Project would be subject to the settlement agreements between DW and the DWR, and between DW and the USBR, to protect the water supplies and senior water rights of the two projects. These agreements effectively preclude diversions to storage by DW when the projects[3] calculate that unappropriated water is not available, as well as preventing discharges by DW when discharges of stored water would require changes in SWP or CVP operations to meet state or federal mandates.

The above factors all affect water availability in some measure. It is uncertain how much water would be available after all of these factors are considered. In particular, CCWD's X2 restriction, plus the restrictions in the biological opinions for the DW Project would especially affect diversions when protected fish are present during the fall and winter and could be harmed by the diversions. The X2 restriction also could restrict diversions during April and May if they were not already restricted for fish protection reasons. Additionally, some of the restrictions in the DW biological opinions would further reduce

[2] See below for a more complete explanation of the X2 restriction.

[3] If the DW Project received water right permits, the permits likely would require that in the event of a dispute between DW and one or both of the projects regarding the availability of water for diversion at a given time, DW would bring the dispute to the SWRCB for resolution.

the opportunities of DW to divert water at times when diversions would be injurious to fish. Accordingly, it is uncertain that adequate water is available for the DW Project.

II. WATER QUALITY CONSIDERATIONS

The effect of the DW Project on water quality is a major concern. This issue has two distinct but not entirely separable components: salinity and organic carbon. Salinity affects water quality directly. Salinity also contributes to the formation of precursors of groups of molecules which include trihalomethanes (THM's) and haloacetic acids (HAA's). As discussed below, the evidence shows that during storage there will be an increase in the concentration of both salts and DBP precursors in the water stored in the DW Project reservoirs. Additionally, the DW Project could, under certain circumstances, divert water to storage containing a higher concentration of salts than the concentration of salts in the channels of the Delta during the months when the DW Project would likely release water.

It is the policy of the SWRCB to ensure in any of its water right or water quality actions, that the policy set forth in Resolution 68-16 is followed. Resolution 68-16 provides in pertinent part:

> "1. Whenever the existing quality of water is better than the quality established in policies as of the date on which such policies become effective, such existing high quality will be maintained until it has been demonstrated to the State that any change will be consistent with maximum benefit to the people of the State, will not unreasonably affect present and anticipated beneficial use of such water and will not result in water quality less than that prescribed in the policies."

Applied to the water right applications for the DW Project, Resolution 68-16 means that before the SWRCB will approve the water right applications, the SWRCB must be satisfied that storage of water on DW Project reservoirs and subsequent releases of water into the Delta either will not adversely affect the quality of water in the Delta channels when it is released, or that any reduction in water quality will "be consistent with maximum benefit to the people of the State, [and] will not unreasonably affect present and anticipated beneficial use of such water...."

It does not appear that the SWRCB can make the required finding. During the hearing, the SWRCB received evidence that the release of stored water from the DW islands is not consistent with the interests of purveyors of drinking water who will invariably receive the water if it is released into the channels of the Delta. As discussed herein, such released water would on many occasions contain elevated levels of salts and organic carbons compared with the receiving water, although the exact amount of added salts and organic carbons could vary. A significant increase in these constituents could substantially increase the costs of treating Delta water during the period when DW releases the water. Because any increases in dissolved organics would result in increased treatment cost during the time when DW releases the water, the most likely users of water from the project, urban water suppliers, have stated that they will not buy the water and do not want to receive it as part of the water delivered by DWR or USBR. With no identified beneficial use for this water, its discharge cannot be consistent with the maximum benefit to the people of the state. Moreover, without additional evidence, it cannot be determined that the resulting degradation will not unreasonably affect present and anticipated beneficial uses of the water in the Delta.

II.A. Dissolved Organics

Organic carbon loading is generally expressed in parts per million of either total organic carbon (TOC) or DOC. TOC consists of both DOC and particulate organic carbon (POC), which includes diatoms and other microalgae, dead algae, bacteria, microzooplankton, and decomposing plant material. For purposes of the DW Project, TOC and DOC are nearly interchangeable: DOC represents more than 90 percent of the TOC present in Delta waters (CCWD 4, p. 10; RT pp. 485 – 486, 1067). Except where specified, the term DOC is used in this decision to include all organic carbon.

Storing water on the reservoir islands would increase the DOC content of the water through leaching of peat soils, return flows from interceptor wells, and growth and degradation of plant material. (DEIR/EIS Appendix C-5, Table C5-3; TR 425 – 426; DW 13, Figure V-5; RT pp. 2779 – 2780.) The issue is whether DW operations will have a significant effect on the quality of the receiving waters when stored water is released, and whether the degradation will be offset during the same time period by the cessation of agricultural practices on the project islands. The parties presented conflicting evidence regarding the amount, nature, and effects of DOC loading that would be caused by the DW Project.

(1) Changes in DOC as a Result of the Project

The water will increase in DOC while it is stored, at least during the early years of the project, because of leaching from the peat soils on the floors of the reservoirs. During the release period (primarily July, August and September) in many years, DOC in released water would be higher than in the receiving water (RT pp. 507 – 509, 2545). There would also be some increase in export water DOC levels during the release period in almost every year, regardless of release rate. If project water were released at the rate of 1000 cfs or 10 percent of the total assumed export rate, the average increase in DOC, taking into account reductions in agricultural loading of approximately 2.0 mg/l, would be about 0.2 mg/l.[4] One proposed mitigation measure (CUWA 2, p. 11) would effectively prohibit any release of water for export unless the water was pretreated.

Some evidence suggests that on an <u>average</u> <u>annual</u> basis the quantity of DOC in export water might decrease because agricultural return flows will decrease or cease on the DW islands (DW 13, p. 23). Other evidence rebuts this evidence. (*See*, e.g., CCWD 10.) Compared to the No-Project alternative (intensive agriculture), cessation of agricultural

[4] This assumes that the loading during storage and evaporation would be equivalent to the estimated present agricultural loading of 2.0 mg/l.

operations on the DW islands could reduce the annual quantity of DOC loading to the Delta (RT pp. 171 – 173).[5] In any event, the timing of the DOC loading would change. DOC loading from agriculture currently occurs primarily in winter. With DW operations, the loading from the reservoir islands would shift to summer. With the higher salinity in the DW reservoir releases (see below), this shift in DOC loading could increase water treatment costs for urban water users.

(2) Loadings of DOC from Initial Fillings

Over the long term, repeated filling and emptying of the DW reservoir islands might leach out most of the soluble organic material. If new plant growth were minimized, annual DOC loading might decline. The first few fillings, however, might have very high levels of DOC, plus residues from pesticides and other island wastes (RT pp. 2549 – 2552). If permits for the DW Project restricted discharges of poor quality water, the opportunities to release these early fillings could be few. One option would be to release water from the islands at a very slow rate,[6] under winter flood conditions, when there is positive (downstream) flow in Old and Middle rivers. (CUWA 2, pp. 11-12.) Under this option, emptying the reservoirs could take several months or even several years. Even with tight restrictions, releases from Bacon Island could still impact the quality of water at the intakes for the SWP and for CCWD in the southern Delta.

Notwithstanding extensive evidence and analysis in the record, including models, hypotheses, and theories about the amount of DOC production when the islands are first filled, none of the predictions regarding DOC loading appears reliable. Depending on the length of time of the initial storage cycles, the reservoir islands could build up substantial loadings of DOC and other constituents that, when discharged, could result in violations of drinking water standards in exported treated water even if the discharge was made under high flow conditions. An experiment on wetland flooding showed a rapid rise to high DOC levels, and while it produced a higher concentration than might be expected in a filled reservoir, it indicates the type of reaction which could occur in the early years of the project (SWRCB 2, Appendix C3, pp. C3-6 to C3-8). Because of the potential for natural disasters involving seismic and storm events in the Delta, it would not be in the public interest to have filled reservoirs in the Delta that would require a slow release; yet a slow release might be needed to avoid impairing beneficial uses of the receiving water for municipal purposes.

(3) Resuspension of Organic Material During Emptying of Reservoir Islands

After the project operated for several years, there might be relatively low concentrations of DOC leached into the stored water from the underlying peat soils, especially when the reservoirs were full. It is reasonable to assume, however, that when the reservoirs are drawn down concentrations of DOC and POC will increase in the remaining water as wind stirs up the bottom. If the water is very shallow, the resuspended material could

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[5] Fertilizer use would also be significantly reduced (RT p. 179).

[6] A discharge rate maximum of 10 percent of Old and Middle rivers flow was suggested during the hearing.

form a slurry of water, peat, silt and other materials (CUWA 10, p. 8). If this water could not be released because of DOC, turbidity, temperature, or other limitations, it could substantially affect project yield. Each foot of water not released from the reservoirs (or not stored), would decrease project yield by approximately 10,000 acre-feet (RT p. 479).

It is likely that there would be at least some resuspension of material, especially in the early years of the project. Even the development of a silt layer in later years would not preclude substantial resuspension in high wind conditions. The evidence shows that some resuspension could occur due to wind fetch on the DW reservoir islands. (RT pp. 2402 - 2404, 952 - 954.) Mitigation could include limits on DOC, turbidity and other relevant water quality parameters in discharged water. Also, installation of barriers, floating curtains, levees or island structures could help reduce fetch size and wave development. Such structures, however, could reduce project storage capacity and yield.

(4) Residence Time

Residence time is an important consideration in estimating loadings of DOC in reservoir water. The water probably would remain on the islands longer than the median length of eight months suggested by DW operational studies, which assumed the highest export frequency, and therefore the shortest residence time. During wet periods, or when export pumping capacity is limited, the water could remain on the islands for several years. Even if the rate of production of DOC decreased over time, evaporation and consequent salinity and DOC increases would continue to occur.

(5) Unwanted Receipt of DW Project Water

Certain municipal water providers would receive, and would have to treat, any incremental increases in DOC that would occur when DW released water, whether they had purchased the water or not. DW discharges could not be isolated from other water diverted at the pumping facilities in the southern Delta. Further, DW discharges could represent a substantial proportion of the total exports from the Delta in late summer. The City of Tracy is the first customer along the Delta–Mendota Canal; the South Bay Aqueduct is the first major distribution branch from the State Water Project's State Aqueduct, and CCWD's Rock Slough intake is located near Bacon Island and "downstream" (under reverse flow conditions) from Webb Tract. CUWA represents numerous urban water agencies that would receive this water if it was discharged into the Delta for export. CUWA's member agencies state that they have no interest in buying this water or in receiving deliveries of water that has been mixed with DW releases.

The only completely effective way to prevent the DW Project from causing increases in pollutants received by the municipal water users would be to require no degradation of receiving water, as proposed by CUWA. If some degradation of receiving water quality were consistent with maximum benefit to the people of the state, as provided in Resolution 68-16, the SWRCB could condition any water right permits on not exceeding an incremental degradation of water quality in the receiving water. In this case, however, there are no willing customers and no overriding public need for the water. On the other side of the balance, there is a potential for harm or substantial expense to municipal water purveyors.

(6) Shallow-Water Habitat and DOC Production

The DW Project includes shallow-water habitat on the reservoir islands when they cannot be filled (RT pp. 259 – 260). The purpose of the habitat would be to provide food and cover for waterfowl for hunting purposes; however, the habitat is not part of the wetlands mitigation requirements.

The evidence regarding the effect on DOC concentrations of creating shallow-water habitat is inconclusive. On the one hand, keeping the soil moist on the reservoir islands, as would occur during storage as well as during the shallow-water flooding proposed during nonstorage periods, would reduce loss of peat soil due to oxidation, and would also reduce release of DOC into the pore water of the peat soil (DW 13, p. 115). On the other hand, the shallow flooding would encourage the growth of shallow water plants (emergent vegetation), which will decompose when the reservoirs are filled.[7] At an average depth of about one foot, considerable aquatic plant growth is likely. Consequently, in the presence of shallow water, the reservoir islands would produce a continuing source of new DOC, at least partially offsetting the benefits of reduced peat oxidation. DOC loading will be less if the DW Project does not grow seasonal wetlands on the reservoir islands. (RT pp. 2568 – 2571; 2812 – 1813.) It might be appropriate to prohibit shallow flooding of the reservoir islands.

(7) Wetland and Plant Degradation Experiments

Much of the information in the draft EIR/EIS on DOC and POC loading that would be caused by the DW Project was based on a series of field and tank experiments. The purpose of these experiments was to estimate DOC release from flooded wetlands, and from degradation of plant material and peat. These experiments were discussed at length in the draft EIR/EIS (SWRCB 2, Appendix C-3). The experiments took place over a short duration (CUWA 5, p. 20), and some experiments were conducted in winter months when the lower temperatures would reduce production of DOC.

The results of the experiments are inconclusive with respect to the proposed project. None of the results are directly related to what would occur on the reservoir islands, especially after multiple years of storage or creation of shallow-water habitat. The DWR is developing new studies on Delta island flooding, but it is not clear that DWR's studies would be helpful in predicting the effects of DW Project operations. (RT pp. 1628 – 1633.) A medium-to-large scale pilot project, or staged development, that addresses these questions would be helpful in evaluating whether DW or a similar project should be allowed to proceed with full-scale development.

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[7] Some evidence suggests that much of the increase in DOC is due to degradation of plant material rather than leaching from peat. Other evidence suggests that more DOC could result from peat than from wetlands. (CUWA 5, p. 19.)

(8)   New EPA Requirements

The United States Environmental Protection Agency (USEPA) is currently reviewing the allowable limits on DBP's. The parties presented testimony indicating that USEPA will require implementation of new Safe Drinking Water Act requirements in the next few years. (*See* 42 USCA § 300f through 300j-26.) The new requirements are expected to reduce the allowable amount of DBP's in drinking water in stages. The first stage was projected to take effect in 1998. The hearing evidence indicates that new, more stringent, requirements than existed at the time of the hearing will be in place before the DW Project could begin operations. The restrictions proposed by USEPA would affect both DBP levels and TOC levels in the untreated water. In addition, removal requirements for urban water purveyors receiving the water will vary depending on the level of salinity in the water. The new requirements are expected to increase treatment costs. If these projected changes in regulatory requirements take place, and the DW Project incrementally increases the amount of TOC and salinity in the source water, the operation of the DW Project would add to the increased treatment costs.

II.B. Salinity Impacts

Salinity is usually measured in electrical conductivity (EC), but some of the evidence presented in the hearing was given in terms of total dissolved solids (TDS) or chlorides. The major ions of concern when discussing salinity are chlorides and bromides. Bromide ions are considered to be more reactive in the formation of THM's. The major source of bromides is ocean-derived salinity. Some bromides may also be returned to the system with agricultural return flows (SWRCB 2, pp. 3C-10 & 3C-11).

Salinity increases in the Delta caused by DW Project discharges could have a significant impact before they reach the level of significance defined in the draft EIR/EIS. The draft EIR/EIS assumes that there will be a significant effect if a DW discharge causes a salinity increase equal to 20 percent of the numerical water quality objective, or if the discharge causes the receiving water to exceed 90 percent of the numerical objective (SWRCB 2, pp. 3C-20 to 3C-21). This level was selected because a combination of natural variations in the system, plus inaccuracies in modeling operational effects, might provide sufficient "noise" in the system that water quality changes of less than 20 percent of the standard could not be unequivocally attributed to the effects of DW operations. Under this approach, a change from 50 mg/L chlorides to almost 100 mg/L would not be considered a significant impact on CCWD, since such a change would be less than 20 percent of the current 250 mg/L chlorides objective at Rock Slough (RT pp. 283-290). This level of change in salts in the receiving water, however, would reduce the benefits to CCWD of having Los Vaqueros Reservoir (LVR). The Los Vaqueros intake is on Old River. LVR was built specifically to divert and store fresher water to blend with saltier water diverted at Rock Slough, to improve quality. Permitting release of water that is saltier than the receiving water could reduce the benefits of the LVR project.

It is likely that some degradation of water quality in the receiving water channels of the southern Delta will occur in at least some years when water is discharged from the DW reservoirs. The draft EIR/EIS model results suggested increases of up to 50 mg/L chloride might occur in export water under Alternative 1 (SWRCB 2, Figure 3C-18) during the discharge period.

(1) Salinity Intrusion

DW's high diversion rates during filling could reduce Delta outflow, allowing the head of the saline wedge of ocean water in the estuary to move farther upstream than would otherwise occur. This movement is measured as an increase in X2, an index value defined in the Monismith equation (RT pp. 349 – 356) as the distance in kilometers above the Golden Gate of two parts per thousand (ppt) bottom salinity. A higher value indicates that salinity has penetrated farther up the estuary. Such a movement could impact the water quality of water used by CCWD, the City of Antioch, and several industries (CCWD 4, p. 6), as well as the state and federal pumps in the southern Delta.

CCWD's senior water rights, as expressed in the permit terms and conditions for operation of LVR, include stricter operational restrictions than those set forth in the federal Biological Opinions (BO's) for the DW operations. Under the federal BO's, DW would, under certain circumstances, be able to divert water to storage when CCWD could not divert water (CCWD 3, p. 11.). In addition, the BO's would allow DW to divert when X2 is farther up the estuary than allowable for diversions to LVR. The process of filling the DW reservoirs could hold X2 at a location where CCWD would be unable to divert to LVR, even though it has senior water rights. Such operations could reduce both the quantity and the quality of water taken at CCWD's intakes.

It is possible that such a circumstance would only occur rarely (RT pp. 1411– 1412), and that the incremental change in salinity (and frequency of CCWD diversions) between west of Chipps Island and Collinsville would not be significant.

DW should not be allowed to divert water during periods when senior appropriators cannot divert water under their water rights unless it can be demonstrated that there would be no adverse effect on senior appropriators. These periods include periods when there are restrictions on diversions to protect sensitive fisheries, test periods, and times when CCWD cannot divert to storage in LVR. As discussed above, DW Project diversions could lengthen the period over which these restrictions prevent CCWD from diverting water.

(2) Quality of Diverted Water

The second salinity issue addresses the quality of the water that DW would divert onto the reservoir islands. DW proposes to fill the islands with surplus flows primarily in the late fall and winter months when storms provide surplus flows (CCWD 4, Figure 9). This water would be released into the Delta for export primarily in July, August and September (CCWD 4, Figure 10). Surplus flows from late fall and early winter storms could include substantial amounts of salts from agricultural runoff, especially from San Joaquin River tributaries (CCWD 4, p. 11), and from salty flushing flows from Delta agricultural fields (CCWD 4, p. 20). Even if there was no subsequent evaporation on the reservoir islands, this water could have higher TDS levels than the receiving water in southern Delta channels, which in late summer is often provided water of relatively good quality released from Sacramento River reservoirs.

In the absence of restrictions on diversions, DW would fill its reservoirs as early in the autumn as allowed, to ensure that the reservoirs are more likely to be filled should the winter turn drier. Terms of the BO's prohibit DW from diverting for some days after the onset of the first winter storm (elevated outflow) to avoid harming outmigrating fish. In addition, the BO's require DW to reduce diversion rates. The early storms carry salts and other chemicals from agricultural lands, which often elevates the salinity of the early flows. Later in the winter, many of the agricultural Delta islands release salty water from soil leaching. These restrictions could delay the onset of filling by DW until water quality improves after a succession of winter storms. A delay should both move X2 farther west (reducing salinity intrusion) and transport salts from agricultural return flows out of the Delta. Depending on the circumstances of a particular year, the requirements in the BO's could result in severe restrictions in DW's operations, or even prevent DW from diverting water.

(3) Evaporation

Water stored on the reservoir islands for later release would increase in salinity through evaporation, especially during the spring and summer. About 35,000 acre-feet of evaporation would occur on the reservoir islands each year (RT p. 278). This would concentrate salts in the reservoir water, and it is unlikely that they could be diluted over the summer months while the water was in the reservoir. DW's final operations criteria prepared for its endangered species consultations contains a topping off provision for June through October that could allow diversion of high quality water onto the islands to dilute the accumulated salts, if water is available during those months. In those months in most years there is no water available for appropriation to storage under the water right priority of DW's applications, however. (RT pp. 277 – 278.)

Stored water might be held on the islands for extended periods. DW assumed that the water usually would be sold and discharged in the late summer after it had been stored the previous winter. The draft EIR/EIS identifies this type of operation as the "worst-case" for determination of environmental impacts. During a succession of wet years, however, or when customers or pumping capacity are unavailable, DW might store the water for several years before being able to sell it. Due to evaporation, the salt load of the stored water would increase over time. Winter topping off could dilute the salinity of the stored water somewhat.

Additionally, DW's plan to operate the reservoir islands as shallow-water habitat during periods when there are not sufficient surplus flows to fill the reservoir islands (RT pp. 259 – 260; RT pp. 2647 – 2650) could add to the salt load discharged to the Delta channels. Shallow-water habitat operations might continue for several years during a drought. For example, model runs show the islands essentially empty throughout the period 1987 – 1991 (CCWD 3, p. 26).

(4) ***Salinity Effects on Water Users***

If the DW Project caused only a small increase in salinity in the Delta channels, it might be possible to dilute the releases enough to make the effects on the end user insignificant. At an expected DW release rate averaging about 4,000 cfs for a month, however, and assuming the total exports in late summer will be 8,000 cfs to 12,000 cfs, water released from the DW reservoirs could amount to 33 percent to 50 percent or more of the total amount of water exported from the Delta. While some water released from Webb Tract might be tidally mixed and not transported to the export pumps, virtually all water released from Bacon Island would be exported, because the export pumps cause the flows to reverse in Old and Middle rivers when there is low Delta inflow. Most of the water released from the DW reservoirs would flow into Clifton Court Forebay. Some could reach the USBR Tracy pumps. The municipalities in the Santa Clara County area, served by the South Bay Aqueduct, would receive the DW water diluted only by the water in the Delta channels.[8] CCWD also would receive this water directly, because its intakes at Rock Slough and Old River are near the DW discharge points. This effect would continue during the time needed to empty the reservoirs, approximately one to two months if there are no restrictions on the discharge rate to control increases in salinity or DOC in the Delta channels.

(5) ***Potential Net Reductions in Salinity Due to Foregone Agricultural Activities***

DW argued that the DW Project would cause a net improvement in salinity in the Delta on an average <u>annual</u> basis because of the cessation of agricultural activities on, as well as return flows from, the project islands.[9] In most years, however, this reduction would be counterbalanced by an increase of salinity in the receiving waters (Delta channels) when water is released from the reservoir islands.

Foregoing irrigation on the project islands would not usually cause an increase in Delta outflow that would improve water quality, as suggested by DW. (RT pp. 306 - 311.) When the Delta is in "balanced conditions", the DWR and USBR release only enough water from upstream reservoirs to maintain the water quality standards. (CCWD 4, pp. 18-20, exhibit 8.) Any saving in Delta consumptive use would either be exported or saved in upstream reservoirs for later use.

III. POTENTIAL PHYSICAL IMPACTS OF THE PROJECT

CDWA, PG&E, and CALTRANS raised concerns regarding levee stability, potential damage to and interference with PG&E's gas lines, seepage impacts, and impacts to State

[8] If the municipalities have regulating reservoirs in their systems, they could further dilute the salts before treatment.

[9] In a comparison of the model estimates of the salinity of agricultural return flows from Bacon Island with actual measured salinity (CUWA 7a; CCWD 8, figures 2-6), the measured values were significantly lower than the modeled values. Therefore, the degree of water quality improvement to be expected as a result of foregone agriculture apparently would be less than DW predicts.

Highway 12. DW argued that the protests of CDWA, PG&E, and CALTRANS[10] were matters raising disputes over real property rights and were outside the authority of the SWRCB to resolve. The evidence indicates that the DW Project could cause property damage to them or to their constituents. While DW might be liable to them if such damage occurs, the bases for the protests by these parties are that by harming these parties DW would be acting contrary to the public interest. Other parties, including EBMUD, also raised some of the following issues concerning the public interest. The following paragraphs address the harm that could be caused to the public interest by the DW Project.

The CDWA suggested that the SWRCB require DW to provide funding and financial security to ensure that neighbors of the project who are affected by it can financially deal with problems caused by the project and can ensure that the project is operated to avoid damage on neighboring islands. While DW has indicated it is willing to put money into a trust fund each year to ensure that operating costs of project mitigation devices are met, such as operating the interceptor wells or emptying the reservoirs in an emergency, DW has refused to offer any kind of surety bonds or other security to pay for property damages on neighboring islands. Damages could occur to PG&E's gas pipeline, EBMUD's water pipelines, railroads, levees, farmland, and other uses of Delta islands. DW should provide information on the surety bonds or other assurances it would be willing to provide to pay for any damage caused by the DW Project.

No statute specifically states that the SWRCB has authority to require financial assurances in cases where protestants may suffer property damage as a result of the SWRCB's action.[11] (D-1587; D-1011.) Nevertheless, the SWRCB has broad public interest authority, and if the SWRCB finds that it is not in the public interest to allow a particular activity unless potential impacts are mitigated, then the SWRCB can condition any permits it issues upon the permittees providing adequate mitigation. (Wat. Code § 1253.) In this case, reasonable mitigation could include a term or condition requiring that the permittee obtain and maintain insurance or other financial assurances adequate to pay for damages caused to neighboring property by the appropriation of water. On the current hearing record, however, there is inadequate evidence to determine the amount of insurance or other financing that would be needed. Additionally, if the SWRCB were to

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[10] The CALTRANS request is to exclude a 100-foot strip of land from conversion to wetlands on the south side of the highway as it crosses Bouldin Island, which is a proposed habitat island. CALTRANS was seeking to avoid having to mitigate for impacts to a new wetland if and when it widens Highway 12. The modification requested by CALTRANS would reduce the amount of land included within the habitat management plan as mitigation for the effects of reservoir storage on wildlife. It does not appear to be in the public interest to reduce the amount of mitigation for the DW Project in this situation. Accordingly, the the restrictions requested by CALTRANS are not recommended for the DW Project.

[11] CDWA cited a statute that provides for financial assurances to guarantee that mitigation measures will be carried out, but it deals with real property development, not water rights. (*See* Gov. Code § 66499 et seq.) Also, the law cited by CDWA assures the <u>county</u> that mitigation will be done. It does not provide assurances to neighboring property owners whose property might be damaged.

issue permits, it would be helpful to receive additional evidence and recommendations regarding mechanisms for administering financial assurances.

III.A. Seepage

The hearing record shows that in the absence of active measures, seepage will occur between the channels of the Delta, the DW reservoirs, and neighboring islands. (SWRCB 2, pp. 3D-8 through 3D-15.) Agricultural uses on neighboring islands could be impaired by seepage-induced flooding or moisture damage from DW reservoirs. (CDWA 14.) Seepage onto Delta islands is a common occurrence in the Delta. (SWRCB 2, p. 3D-4.) Two kinds of seepage occur: "high" seepage passing through or immediately beneath a levee and "deep" seepage passing through permeable materials beneath the peat and silt that underlies most Delta levees. (SWRCB 2, p. 3D-3.) High seepage accounts for wet places near levees, and comes from the adjacent Delta channel. Deep seepage causes wet areas on the interior of an island. The typical practice in the Delta is to capture seepage in interceptor trenches or relief wells on the islands near the levees and pump the seepage back into the adjacent channel. (DW 17, pp. 7-8.)

The draft EIR/EIS describes the dynamics of deep seepage as follows:

> "The amount of seepage that occurs is controlled by the permeability of soils, length of the seepage path, and height of the hydraulic head (i.e., the pressure created by water within a given volume). The problem is worsened in the Delta by the decline in the level of peat soils, which increases the hydraulic head between channel water surfaces and the islands, and by the presence of permeable subsurface sand layers. Seepage has been reported to increase after flooding of an adjacent island and to cease after the flooded island has been drained." (Citations omitted.) (SWRCB 2, p. 3D-3.)

Two kinds of adverse effects could result from seepage associated with the DW Project. First, there could be impacts to agriculture on neighboring islands due to seepage from the DW reservoirs while they are storing water. This seepage could cause property and crop damage. (CDWA 13 & 14.) Second, there could be seepage from the channels of the Delta onto the DW islands when the water elevation in the reservoirs is less than the water elevation in the surrounding channels. This seepage could result in DW collecting water to storage either outside DW's authorized diversion season or during periods when there is no water available for appropriation under DW's water right priority. If the water came from the channels in the Delta, DW could deplete water for which a senior water right holder has a claim. In this circumstance, DW could be illegally diverting water.

The DW Project reservoir islands and most of the islands in the central part of the Delta are underlaid by a single sand aquifer. (RT p. 130.) The aquifer ranges from 20 to 50 feet thick. Water will move through the sand aquifer in response to hydrostatic pressure on the aquifer.[12] When an island reservoir is filled with water, there will be hydrostatic pressure on the bottom of the island. Because the bottom of the island is permeable, the pressure will also be applied to the aquifer. Water in the aquifer will flow at a rate proportional to the hydraulic head. (SWRCB 2, pp. 3D-2 through 3D -3.).

DW proposes to control seepage and maintain "no net seepage impact" by installing up to 900 interceptor wells drilled through the reservoir island levees, on about 20 miles of the 26.6 miles of perimeter levees encircling the two reservoir islands. (DW 17, p. 9; DW 62, p. 7.) A 1991 estimate by DW assumed that the interceptor wells would be spaced 160 feet apart, be 50 feet deep, and discharge 70 gallons per minute. There is evidence in the record that a much closer spacing would be needed. The purpose of the interceptor well system would be to maintain the water table under the islands at or near the water table elevation in the absence of a filled reservoir. The theory is that the wells would eliminate hydraulic forces that would cause seepage on adjacent islands. DW would put monitoring wells on neighboring islands to assess the effect of the reservoirs on the neighboring islands. DW proposes to return the intercepted seepage water to the reservoirs. (SWRCB 2, pp. 3D-8 and 3D-9.)

To test the concept of using interceptor wells, DW's consultant conducted a demonstration project on McDonald Island, which is receiving seepage from Mildred Island (flooded since 1983). The wells lowered the underground water level during the test, making it possible to run a light tractor on the fields, but the test was conducted over a relatively short period. Even during the short testing period, some test wells experienced difficulties. (CDWA 13 & 14; see also SWRCB 2, p. 3D-10.) CDWA's witness, whose land on McDonald Island was used for the test, testified that while the seepage was reduced within a few feet of the wells, and a light tractor could be driven over his land, his land remained unfarmable because the farming equipment that would be pulled by a tractor would become stuck. (CDWA 13 and RT pp. 796 - 809.) The test differed somewhat from DW's current proposal, since the wells were drilled on the adjacent island, not through the levees on the flooded island. (CDWA 13.) As discussed below, it is uncertain whether the Division of Safety of Dams would authorize DW to perforate its reservoir levees with up to 900 interceptor wells, as this might weaken the levees.

DW proposed monitoring, along with a significance standard for determining whether seepage onto neighboring islands merited action by DW, in addition to proposing seepage control measures. CDWA presented testimony to show that the significance standard and the proposed seepage control system are not adequate to protect CDWA's members. (CDWA 13, pp. 3-4.) Evidence regarding routine seepage problems in the Delta and the effects of the test calls into question the effectiveness of DW's plans for monitoring and seepage control measures. (DWR 24, CDWA 8.) The effectiveness of this type of

[12] This is an expression of Darcy's Law: The amount of seepage that occurs is controlled by the permeability of the soils, length of seepage path, and height of the hydraulic head (i.e., pressure created by water within a given volume.) (See also, SWRCB 2, p. 3D-3.)

system on a large scale has not been demonstrated and does not address the varied soil conditions that exist in the Delta. (CDWA 13, p. 3.) Other potential methods of seepage control are mentioned in the draft EIR, but some would require easements on neighboring islands. The record contains no indication that DW has, or could obtain, easements on neighboring islands except for the limited access DW obtained during the McDonald Island demonstration project.

Seepage from the reservoir islands could be exacerbated if DW carried out its proposal to obtain construction material from the reservoir islands to raise and widen the levees. DW proposed to obtain sand from borrow pits on the reservoir islands to use as construction material.[13] Taking material from borrow pits would involve removing a blanket of silt and peat about 10-15 feet deep from the floor of the islands at each borrow site, to reach the sand. The peat and silt layer impedes percolation of water, and with it removed, exposed areas in the borrow pits could be subjected to as much as 24 feet of hydraulic pressure. (SWRCB 2, p. 3D-13.) This could increase the seepage rate to adjacent islands. Conversely, when the reservoir islands are empty, water could enter the islands under pressure by way of the borrow pits.

The draft EIR (SWRCB 2 at 3D-11) suggests that where seepage restrictions are needed, a 2000 foot setback from the final toe of an improved levee is the closest excavation that should be allowed for purposes of obtaining levee materials. The 2000 foot setback would apparently be required for all borrow sites, since seepage sites cannot be adequately identified prior to filling the reservoirs. (CDWA 13.) Because of concerns about the feasibility of DW's interceptor well system proposal, financial assurances also would need to be required to pay for any damage to farming on adjacent islands as a result of seepage from the DW reservoirs. DW objected to providing financial assurances.

The interceptor well system would have to operate whenever water was in storage, even during power failures. With the potential for seepage, it also would have to operate even if DW abandoned the project with water remaining in the reservoirs. Maintenance and operation of the seepage control system by others if the project were abandoned could unfairly burden other parties. Accordingly, a way to assure payment of these costs is essential to the project.

Additionally, since DW intends that the proposed interceptor wells would return any water they pumped to the island, the interceptor wells could divert water from the sand underneath the Delta channels and levees outside DW's diversion season as well as catching seepage. Such diversions could affect the flow of water in the channels of the Delta. In the summer, the water diverted likely would be water from CVP and SWP storage upstream of the Delta. To avoid an illegal diversion, it would be necessary to require the DW Project to avoid any increases in storage outside its diversion season. In

[13] It is not clear whether DW plans to use earth moving equipment or hydraulic dredging to move the material. If hydraulic fill is used, it may not be stable.

most years, this would mean that DW could not divert water during the summer to compensate for evaporation and would have to discharge water to the Delta channels from the interceptor wells.

The SWRCB has previously held that it can deny or restrict a project if the hearing record contains substantial evidence showing that property damage is likely and that it would be contrary to the public interest to authorize the project in light of the damage. (SWRCB Decisions 1523; 1280.) Additional support for this position is provided in the Water Code and in case law. (Wat. Code §§ 1253, 1255, 1256; *Johnson Rancho County Water District* v. *State Water Rights Board* (1965) 235 Cal.App.2d 863 [45 Cal.Rptr. 589].) Accordingly, the SWRCB may, in the public interest, prevent potential damages to neighboring landowners by denying the applications, requiring financial assurances, or requiring additional measures to ensure the stability of the facilities.

III.B. Levee Stability

Levees in the Delta are used to prevent inundation of the islands and serve to define the channels in the Delta. Many levees are fragile. The draft EIR/EIS includes the following statements about the Delta levees:

> "The Delta levee system initially served to control island flooding. Today the levees are necessary to prevent inundation of island interiors during normal runoff and tidal cycles because island interiors have been lowered by extensive soil subsidence. [...] Delta lands have historically subsided at rates that are among the highest in the world." (SWRCB 2, p. 3D-2.)
>
> "Levee failures occur as a consequence of overtopping or levee instability." (*Id.*)
>
> "Factors contributing to levee instability include seepage, settlement, erosion, subsidence, and seismicity." (SWRCB 2, p. 3D-3.)

Delta levees are highly important, both for flood control and to safeguard the export water supply of the SWP and the CVP. In the Delta Flood Protection Act, enacted in 1988, DWR was allocated $12 million per year until January 1, 1999, to develop two programs: the Delta Levee Maintenance Subventions Program and the Special Flood Control Program. (DWR 25, p. 40.) In particular, the Special Flood Control Program requires protection of the towns of Walnut Grove and Thornton and the following islands: Bethel, Bradford, Holland, Hotchkiss, Jersey, Sherman, Twitchell, and Webb. The eight islands are considered critical to the protection of water quality in the Delta, and breaching the levees on any of the eight islands would allow salinity intrusion. (DWR 25, pp. 40-41.) Some of the measures DWR is considering for the islands include rehabilitating levees using imported material and upgrading the levees to the standards contained in DWR Bulletin 192-82, *Delta Levees Investigation* (1982). (DW 24; DW 25, p. 41.)

DW had not prepared a complete engineered design for the DW reservoir levees before the water right hearing. Without a detailed design to focus on, parties in the hearing raised numerous concerns as to the stability of the project levees and the potential for the DW reservoir perimeter and interior levees to fail or be overtopped. (CDWA 13.) Evidence addressed expected weather and seismic conditions, the potential effects of the proposed interceptor well system on levee structural integrity, and the methods to be used for levee construction and maintenance. (SWRCB 2, pp. 3D-3 through 3D-4.) Based on the evidence, an inadequately constructed, maintained, or protected reservoir levee could suddenly crack or gradually erode, causing damage to property on neighboring islands.

Because other agencies have authority to approve dams and levees for large projects, the SWRCB is not obliged to conduct a detailed examination of the engineering aspects of the DW reservoirs. The SWRCB's regulations do not require the applicant to provide an engineered design in connection with an application to appropriate water. Nevertheless, the SWRCB can order a permittee to obtain approvals of the levees from the agencies that regulate dams and levees.

The structural safety of the perimeter and interior levees would be regulated by the Division of Safety of Dams (DSOD) of the Department of Water Resources if the reservoirs could be filled to 4 feet above mean sea level or higher. (Water Code § 6004(c).) DW proposes to fill the reservoirs to 6 feet above mean sea level. Additionally, the perimeter and interior levees are subject to permitting by the USACE and possibly other agencies. Although the SWRCB would not itself need to address the engineering aspects of the levees if it were to permit the DW applications, the SWRCB would require DW to obtain all required permits and approvals from other agencies and would consider requiring DW to obtain, at DW's expense, an evaluation of all of its levees from the DSOD regardless of whether the DSOD's approval is statutorily required.

III.C. PG&E Lines

DW has not made arrangements with PG&E to ensure that PG&E's natural gas pipeline, which crosses Bacon Island, will be protected and will remain accessible to PG&E at times when Bacon Island reservoir is storing water. As discussed above in connection with the seepage issues, the SWRCB has authority to decide whether or not it is in the public interest to allow DW to appropriate water to storage on Bacon Island. If the SWRCB finds, based on the evidence, that the appropriation will present a substantial threat to PG&E's ability to serve natural gas users, the SWRCB can condition a permit or deny an application.

Two primary concerns were raised regarding the gas pipeline. First, is it in the public interest to allow the gas pipeline to be flooded, and if so, under what circumstances? The SWRCB is required to take into consideration the public interest when deciding whether to approve water right applications, and shall reject an application when in its judgment the appropriation would not best conserve water in the public interest. (Wat. Code §§ 1253, 1255.) The gas pipeline involved is one of PG&E's main lines for connecting underground gas storage to users in northern California. This pipeline can transport approximately one-third of PG&E's total system capacity of gas when gas is withdrawn from storage at the maximum rated. (PG&E 2, p. 2; SWRCB 8, pp. 64-65.) It is not in the public interest to allow DW to make the gas line inaccessible to emergency maintenance; nor should it be flooded without protective measures to ensure the integrity of the gas pipe.

Additionally, the SWRCB notes that there is a real property access issue between DW and PG&E because of easements held by PG&E to run its gas pipeline across Bacon Island. The SWRCB is not the proper forum to decide whether or not the applicant or the protestant has the right to occupy or use land or other property necessary to the proposed appropriation. (SWRCB Decision 1516.) This limitation is explicitly set forth at California Code of Regulations, title 23, section 777. Accordingly, while the SWRCB has jurisdiction to authorize the diversion of water to storage on Bacon Island, such authorization could not be adequate by itself to authorize DW to flood the parts of Bacon Island where PG&E's gas pipelines are buried. This is a property ownership issue between DW and PG&E that should be resolved between the parties in court if they are not able to resolve it through negotiation. Any permits the SWRCB issued to store water on Bacon Island would be conditioned to avoid authorizing flooding that would illegally impact PG&E's property rights. Additionally, the SWRCB could defer any actions until the property rights are judicially determined.

IV. PROJECT FEASIBILITY

The SWRCB is required to condition any permit it issues to best develop, conserve, and utilize in the public interest the water to be appropriated. (Wat. Code § 1253.) If the SWRCB finds that a proposed appropriation will not best conserve the public interest, it is required to reject the application. (Wat. Code § 1255.) Numerous factors are relevant to a determination of public interest. In the hearing on the DW Project, the relevant factors raised by the parties included water quality impacts on domestic water supplies, lack of a market for the water, financial feasibility of the project, feasibility of constructing levees and seepage control facilities to contain the water, potential damage to neighboring property, and impacts on fish and wildlife. The SWRCB has broad discretion to decide whether a proposed project would best conserve the public interest. (*Bank of America N.T. & S.A.* v. *SWRCB* (1974) 42 Cal.App.3d 198 [116 Cal.Rptr. 770]; *Johnson Rancho County Water District* v. *State Water Rights Board* (1965) 235 Cal.App.2d 863 [45 Cal.Rptr. 589].)

DW presented testimony to the effect that it could break even on the project if it could yield an average of 160,000 acre-feet of water per year. As discussed below, it is unlikely, with the additional terms and conditions that would be needed to protect other legal users of water, provide seepage control, and minimize impacts from dissolved organics, that the DW Project could yield this quantity. Further, there is not always capacity at the pumps in the southern Delta to move the water DW would develop. Finally, it is unlikely that DW would be able to sell water during wet years because there is less demand in wet years, and any needed supplemental water supplies could be bought at lower prices.

To approve the DW Project, the SWRCB must be satisfied that the project is adequately designed, mitigated, and assured of having a profitable market for its water supply so that it will continue to operate for the expected life of the project. It would not be in the public interest for the DW Project to be abandoned after construction. If the DW Project failed and was abandoned, some of the adverse impacts that would be avoided or mitigated by an operating project could occur. If the reservoir islands were not operating, but nevertheless filled with water from subsurface seepage or from levee breaks, they could cause seepage onto neighboring islands and could present a danger of impacting neighboring islands as a result of levee breakage. They also could contribute DOC to Delta waters.

Without a source of income from selling water, DW might not be able to maintain expensive mitigation measures such as the proposed active seepage control measures and levee maintenance. Further, the benefits upon which any finding of overriding consideration under CEQA might be based would not be realized, making the approval action vulnerable under CEQA.

Approving the DW Project would make the water unavailable to subsequent water right applicants until the permits were revoked. If the permits were held unused for a period of years, they could discourage other applications from being filed.

IV.A. Availability of Conveyance Capacity for Export of Water

To be financially feasible, the DW Project needs to convey water to a place or places of use south or west of the Delta, using the export pumping and conveyance facilities of the CVP or the SWP. If there is no conveyance capacity or inadequate conveyance capacity available to DW, or if DW is unable to make arrangements to exchange water with the CVP or SWP at the times when DW has a customer, DW will not be able to sell its water for export. If it cannot sell its water and export it, the project is not feasible.

DW based its assumptions regarding conveyance capacity on the historic availability of conveyance capacity at the CVP and SWP export facilities. Currently, it is assumed that from July 1 through October there is some capacity available at the SWP facilities to wheel water. DW assumes for the purpose of estimating feasibility, that it will be able to sell and deliver water for up to 50 years. The existing capacity, however, has constraints, and less capacity could be available in the future as the demands of the SWP water users increase.

The chiefs of operations for both the USBR and the DWR testified that conveyance capacity may not be available in the future, and that neither project has previously entered into the type of long-term wheeling agreement that would be needed to assure that DW's water could reliably be wheeled. (RT pp. 1524; 1587.) Both projects already have set priorities for allocating wheeling capacity, and DW's wheeling priority would come after the existing priorities. The USBR chief of operations stated the USBR has virtually no available capacity at any time. DW's expert agreed with this assessment. (RT p. 1525.) DWR presented testimony that (1) there is limited surplus capacity available in the state facilities, (2) there were no negotiations underway for DW to secure wheeling capacity, and (3) DWR was not in a position to guarantee DW it would have wheeling capacity. (RT pp. 1587 - 1588.) As the demands of the SWP contractors increase, DWR will deliver more water to its contractors from its existing unused water rights, further reducing the capacity for other wheeling. (RT p. 1653.) Additionally, DWR will deliver purchased water for its contractors in preference to other water. The DW Project should make a better showing that the water developed by the project can be wheeled reliably. Such a showing could take the form of a contract with the DWR or the USBR, plus an estimate of the frequency and amount of water that would be wheeled.

IV.B. Project Yield With Mitigation

DW's witness stated during the hearing that the lender for the project would not agree to a yield reduction that would drop the project yield below approximately 160,000 afa. (RT p. 2333.) The current estimate is 154,000 afa, which DW considers to be essentially the same as 160,000 afa for purposes of financial feasibility.

Originally, the DW Project yield was estimated at 235,000 afa. The current estimate of 154,000 afa (RT pp. 2334 - 2335) is based on an assumption that any approval of the project issued either by the SWRCB or the Corps will impose no additional terms and conditions that affect yield over and above the impacts of the mitigation measures in the draft EIR and the impacts of the federal biological opinions. The SWRCB commonly imposes conditions on a project in addition to the mitigation measures that may be recommended in an EIR, to address matters that involve other water rights and the public interest. In this case, the SWRCB would likely add a number of terms and conditions that could have a substantial, undetermined impact on project yield. The issues that could result in such terms are discussed in greater detail in other parts of this decision, and they include (1) protection of CCWD's senior water rights by ensuring that operation of the DW Project does not cause the location of the X2 salinity line to move upstream to a point where CCWD cannot divert water under its own rights during its diversion season; (2) constraints to protect water quality in the receiving water; (3) control of seepage between islands; (4) avoidance of inadvertent diversions to storage outside the diversion season by pumping from interceptor wells; (5) levee design and construction and the likelihood of storing water at plus 6 feet above mean sea level;[14] (6) the feasibility of the project after a complete analysis of yield that would take into account the likely level of demand for the water and the probable conveyance capacity in addition to modeling the presence of water in the Delta; and (7) measures to protect the PG&E gas pipeline. The DW yield estimate does not take into account any additional terms and conditions that would be imposed to address these issues. A further analysis would be needed if these items are addressed in the future, to determine whether the project is feasible.

[14] A lowering of the maximum water storage elevation to plus 4 feet above mean sea level would reduce the project yield by approximately 20,000 afa (RT p. 2579.)

Appendix D

Revised Scope of Work and Response to DW Comments by URS Greiner Woodward Clyde

URS Greiner Woodward Clyde
A Division of URS Corporation

500 12th Street, Suite 200
Oakland, CA 94607-4014
Tel: 510.893.3600
Fax: 510.874.3268
Offices Worldwide

July 6, 1999

Ms. Aimee Dour-Smith
Jones & Stokes Associates, Inc.
2699 V Street, Sacramento, CA 95818-1914
Ph. (916) 737-3000
Fx (916) 737-3030

Subject: Geotechnical Scope of Services in Support of the Supplemental EIR/EIS, Delta Wetlands Project, California

Dear Ms. Dour-Smith:

In accordance with our meeting on April 29, 1999 and the review of the State Water Resources Control Board's (SWRCB) letter dated November 25, 1998 on the subject project, we have prepared a technical scope of work addressing the geotechnical issues requiring responses and/or further development. The scope of work prepared below was also based on our understanding the geotechnical issues from our review of the work preformed by Harding Lawson Associates (reports prepared from 1988 to 1992), the project EIR/EIS main volume and appendices, and review or other pertinent literature to the Bay Delta area from DWR, USACE, and published technical papers.

The outline of the proposed scope of work was developed along the content of Attachment A of the SWRCB's letter, specifically related to Item "III.A Seepage" and Item "III.B Levee Stability".

Further, in response to comments at the June 14, 1999 Scoping Meeting, the proposed work has been divided into two phases. Phase 1 includes Task 1, the review of existing data, which initially included a review of the existing data only. Task 1 is now expanded to include a review of the solutions prepared as part of the EIR/EIS and assess their adequacy in relation to the Board's comments. Depending on the adequacy of the solutions in the geotechnical reference documents to the EIR/EIS, revisions (if deemed appropriate) will be made to the scope of Phase 2 as presently proposed. Phase 2 includes all other tasks, but their scope is now considered preliminary and subject to revision following Phase 1.

1. REVIEW EXISTING DATA AND SOLUTIONS, AND REVISE SCOPE OF LATER TASKS AS NEEDED

Background Information:

Harding and Lawson Associates (HLA) had conducted extensive subsurface soil investigation and groundwater monitoring on the Delta Wetlands project site from 1989 to 1992. We have performed a cursory review of these documents for the preparation of this scope of work and will review them in more detail in Task 1. We will also include in our review the on-going work by the California Department of Water Resources (DWR) on the Delta levees investigation and evaluation, the USACE's levee investigations, surveys, and flood damage repairs reports, and CALFED's levee integrity subcommittee released reports. Other relevant published papers (i.e. UC Davis, UC Berkeley) will also be reviewed and used to supplement data needed for the proposed analyses discussed below.

As previously noted, we have developed the proposed scope of our work in seepage and slope stability primarily on the basis of the SWRCB comments on the EIR/EIS of November 25, 1999. It has been pointed out by the applicants that they believe that some of the issues questioned in that letter had actually been adequately addressed in the EIR/EIS. Therefore as a part of the review of background documentation, we will contact Mr. Ed Hultgren and have him point out issues questioned by the SWRCB that to his understanding are addressed adequately in the existing documentation. Based on these discussions, we will specifically review those parts of the documentation, and relate them to the SWRCB's comments. Based on this review, we will adjust the scope of the recommended Phase II supplemental studies if deemed necessary.

Scope of Work:

As discussed, we will review the existing project documentation, primarily in view of establishing data bases for the following work, judging the adequacy and completeness of the past work, and adjusting the scope of the Phase II proposed studies. As a part of this review, we will meet with Mr. Ed Hultgren to obtain his input. The review of the background documentation will be used to:

- Evaluate subsurface soil and groundwater data collected for the project and other relevant documents,
- Review and evaluate the geologic profiles and cross sections proposed for the various analyses,
- Collect and assign material parameters and properties to support the seepage analyses and the levee stability analyses for both static and seismic conditions,
- Review and, if necessary, revise the scope of the following Phase 2 studies,

We will attend three meetings in Sacramento, (1) one meeting with Mr. Ed Hultgren and the project team members to receive and discuss the proposed geotechnical studies relevant to the Board's comments, (2) one meeting to report back to the project team on our findings and evaluation of its adequacy, and (3) one meeting to present the revised scope of Phase 2.

Based on our cursory review of the available data, we do not anticipate to perform additional field exploration and laboratory tests. As discussed, the remainder of the scope of Phase 2 is preliminary and subject to change based on the results of Phase 1.

2. PERFORM SEEPAGE ANALYSES

2.1 Interceptor Wells

Background Information:

Active interceptor or relief wells are proposed for mitigating potential seepage impact on the neighboring islands as a resulting of filling the reservoir islands (Webb Track and Bacon Island). Field groundwater drawdown programs were conducted by HLA on the McDonald Island in 1989-1990 (Phase I) and 1990 (Phase II). The McDonald Island is located adjacent to the Mildred Island that was flooded at the time of demonstration. The field test was conducted to evaluate the feasibility and effectiveness of the interceptor (relief) wells in lowering the hydraulic head in the sand aquifer. HLA (1991) also performed groundwater numerical modeling to simulate various systems of interceptor wells and the required rate of discharge (groundwater withdrawal) that would maintain the existing groundwater conditions at the neighboring islands.

An independent evaluation of the effectiveness and the active interceptor wells will be conducted to provide response to the SWRCB concerns about their adequacy. The activities proposed under this task will include:

Scope of Work:

- Review the test data and conclusions made for the field drawdown program for use as a calibration to the numerical seepage model,
- Develop a baseline condition for the groundwater at the project site (part of Task 1 scope of work). This baseline condition represents the existing groundwater and seepage condition before the installation of interceptor wells and will be used to measure the effectiveness of any proposed well system,
- Develop numerical models and analytical procedures for the groundwater withdrawal simulation,
- Reconcile soil and groundwater parameters used in analyses with the field data (calibration),

- Perform sensitivity analyses,
- Evaluate the interceptor well system (diameter, spacing, depth, screened length) proposed by DW to maintain the existing groundwater condition at the affected islands during high storage in the reservoir islands, and develop recommendations for an optimal system,
- Address the variation in subsurface soil conditions at the project site and its effects on the interceptor well system.

We will utilize computer program SEEP/W (Geo-Slope, 1998) for these analyses. SEEP/W is a two-dimensional finite element computer program used to model the groundwater flow through the porous media. The program is capable of running both steady state and transient time-dependent analyses.

2.2 Effectiveness of Monitoring System and Procedures

Background Information:

HLA developed a monitoring system for groundwater seepage. The monitoring system provides a standard of performance against which project related seepage can be determined.

Scope of Work:

We propose the following subtasks:

- Review the proposed standard monitoring procedures developed for the Delta Wetlands project,
- Assess the adequacy and effectiveness of the proposed procedures to monitor the project related seepage to the neighboring islands,
- Determine volume and time-dependent variation of seepage under various groundwater and subsurface soil conditions (sensitivity analyses),
- Evaluate the monitoring procedure proposed by DW using results of the sensitivity analyses. The existing monitoring procedures may be used and expanded to incorporate analysis findings,
- Evaluate the criteria (termed "significance standard") developed by DW to determine whether seepage onto neighboring islands merited action by DW. Develop alternate criteria as needed, including easily usable tools such as plots and/or tables of correlation among various groundwater parameters, and set thresholds for different levels of response, including reporting to various agencies and the needs for emergency response.

2.3 Routine Maintenance of Interceptor Wells

Background Information:

The SWQCB has expressed concern about the long-term reliability of the proposed extensive groundwater pumping, especially in view of some difficulties reportedly encountered during DW's demonstration project.

Scope of Work:

We propose the following subtasks:

- Evaluate the long-term reliability of the selected well system including its power supply.
- Estimate, using the models developed in Task 2.1, the effects of various plausible pumping outages
- Develop routine monitoring procedures to identify and respond to outages or lack of performance of individual wells, well groups or the entire system.
- Develop routine maintenance procedures/guidelines for the selected system.

2.4 'Unauthorized' Water Diversion into the Storage Islands through Seepage

Background Information:

The SWRCB is concerned that during certain water level conditions in the storage islands and the adjacent channels the pumping from the interceptor wells or direct seepage may constitute "unauthorized" water diversion into the storage islands. A methodology is needed to prevent or account for such unauthorized diversions. We propose to conduct the following analyses to assess this potential impact.

Scope of Work:

- Perform analyses to simulate the potential of inverse flow of the channel water into the proposed storage islands using the groundwater numerical models developed in Task 2.1
- Utilize the hydrograph of the channel water to quantify the seepage into the storage islands at various times during the year and under various groundwater conditions,
- Estimate seepage flow into the storage islands during and outside the DW diversion seasons,
- Use the rate of pond water evaporation consistent with the hydrologic model and incorporate the results into the analyses.

2.5 Effects of Borrow Pits on Seepage

Background Information:

The project proposes to utilize borrow material from the storage islands to strengthen the island's levees. The SWRCB has expressed concern that the limitations on locations of borrow pits proposed by DW may not be adequate to prevent excessive seepage increases in the underlying sand aquifer due to the borrow pits. We propose to perform the following seepage analyses to assess this condition.

Scope of Work:

- Assess the feasibility and effects of borrow pits on the seepage conditions using seepage numerical models developed in Task 2.1,
- Evaluate the proposed size, depth, and setback locations of borrow pits, and make recommendations on an optimal system.

2.6 Settlement Caused by Filling and Pumping of Water

Background Information:

Rapid filling of the storage islands with water causes additional stresses on underlying soil layers. Groundwater pumping from under the levees also causes additional soil stresses. Both of these factors may cause additional settlements of levees and interiors of both storage islands and adjacent islands. This issue appears not to have been addressed in detail. For levee design as well as overall impacts on the project, such settlements should be addressed.

Scope of Work:

- Estimate changes in stress conditions at locations of concern, both periodically changing and permanent. Locations of concern are expected to be the levees, and the interiors of both storage and adjacent islands.
- Estimate the associated settlements and their time histories.

3. LEVEE STABILITY

3.1 Levee Strengthening

Background Information:

HLA (1989, 1992) performed geotechnical investigation and engineering analyses for the Delta Wetlands levees. The study included field investigation, soil laboratory testing, analyses of embankment stability, construction sequence, settlement and seepage through the dam. Design criteria for the levees were also prepared for the California Department of Water Resources, Division of Safety of Dams (DSOD) permit approval.

As part of this study, HLA also developed site-specific static and dynamic soil properties by conducting geophysical surveys and laboratory testing. In addition, HLA developed seismic design load criteria and performed one-dimensional site response analyses. Liquefaction potential evaluation and seismic-induced deformation analyses were also performed.

More recently (1998), probabilistic seismic hazard analysis and levee failure probabilistic evaluation were conducted for the Sacramento/San Joaquin Delta levees by the Seismic Vulnerability Sub-Team of CALFED's Levees and Channels Technical Team. In this study, the delta region was divided into four groups based on their expected seismic ground motions and the levee fragility to failure. Estimates for levee failure due to scenario earthquake events from nearby dominant seismic sources were also developed.

The SWRCB identified various issues associated with the stability of the Delta Levees which included subsidence, static and seismic stability and deformation, settlement, erosion, and overtopping. Although additional work addressing these issues was not requested in the November 25, 1998 letter, during subsequent meetings between the lead agencies and the engineers it was decided that additional engineering analysis of these items was required. Accordingly, we propose to perform the following activities to verify compliance with the state regulatory agencies on reservoir stability issues.

Scope of Work:

- Evaluate the proposed strengthening design for the delta levees,
- Evaluate analysis results from previous studies on the levee stability, including soil engineering parameters used,
- Assess various assumptions on the subsurface soil and groundwater conditions,
- Update dynamic soil parameters based on recent findings,
- Review the various ground motion studies conducted for the Delta Wetlands project

- Develop site-specific seismic load for the project that include peak ground acceleration, design response spectra, shaking duration and acceleration time histories.
- Evaluate the analyses done by DW of the safety of the strengthened levees against static and earthquake induced loads. Implement additional analyses as deemed needed. Failure mechanism should include slope failure, inadequate bearing capacity, excessive slope deformations and settlement, critical seepage conditions and others,
- Evaluate the during-construction, long-term and rapid drawdown static stability of the levee systems, and compare the stability parameters to existing conditions,
- Evaluate the maximum pond water elevation proposed by DW for a safe operation of the reservoir, and recommend a different elevation if needed,
- Evaluate geologic hazards associated with earthquake event, such as liquefaction, loss of bearing capacity and dynamic soil compaction,
- Address the potential for levee overtopping during a seismic induced seiche,
- Address erosion by wind fetch and wave runup,
- Address the constructibility of the selected levee system. We will evaluate the volume and gradation of the materials used to strengthen the levees (see Task II.5 for borrow pits).

For the static stability analyses of the levee systems, we will use limit equilibrium computer programs such as UTEXAS-3. For the seismic evaluation of the levee systems, simplified procedures such as the Newmark sliding block and Makdisi & Seed procedure will used to estimate the expected earthquake induced deformation. We will also run one cross-section using a non-linear 2-D finite element model to validate the calculated deformations from the simplified procedures.

Dynamic soil properties and characteristics (i.e., shear wave velocity and the degradation and damping curves) will be developed using the results of the available studies on similar soils. These studies include: HLA (1992), Stokoe et al. (1994), Kramer (1996) and Boulanger et al. (1998). The selected design seismic loads will be used as the inputs to these analyses.

The stability analysis procedures and criteria proposed in this task will be discussed with DSOD for review and approval. We anticipate a one meeting with DSOD to discuss this matter.

For purposes of the cost proposal, this task has been divided into static aspects and dynamic aspects of the review of stability.

3.2 Assess Effects of Interceptor Wells on Levee Stability.

Background Information:

As several interceptor wells will be installed in the levees, the impact of the construction of the wells on the levee structural integrity will be evaluated. Input from DSOD will also be sought as well as guidelines for installing dewatering wells along levees.

Scope of Work:

We propose the following subtasks:

- Review the practice of construction and operation of water on levee systems as a precedent. We will also performed simplified stability analyses to evaluate the impact of the wells on the structural integrity of the levees.

4. REPORT

We will document the completed work and its results and conclusions in a technical report. As an alternative, we can report separately on seepage and stability aspects. The report(s) will first be submitted in draft form and will subsequently be revised in response to comments by you and the agencies.

OPTIONAL TASKS

Optional Task 1 - Assess Potential Damage to Neighboring Island in Event of DW Levee Breach or Project Abandonment

Background Information:

The SWQCB has expressed concern about potential damages to adjacent islands in the event of a levee failure of a storage island and in the event of project abandonment by the owner. Some effort to address these concerns, using various plausible scenarios, appears justified.

Scope of Work:

We propose the following scope for these contingency events:

- Formulate scenarios of levee failure of storage islands that might damage adjacent islands; one example is levee failure with full storage island (elevation +6 feet) and extremely low water in the channels.

- Estimate potential effect of these event on levee across a narrow channel, and judge the likelihood of any catastrophic damages (some erosion damage could easily be repaired after the event)
- Formulate scenarios of levee abandonment at critical times in the storage islands' annual use cycle, or after a damaging event such as an earthquake (but that does not cause a levee failure)
- Estimate storage islands' behavior for these scenarios, and seepage conditions that could negatively impact adjacent islands; estimate the potential for significant short- and long-range damages to adjacent islands.
- Work with Jones and Stokes' hydraulic modelers to estimate the probability that these conditions could happen.

Optional Task 2 - Attendance at Project Meetings

As a second optional task, attendance by three URSGWC personnel at two project meetings is included in the scope.

Optional Task 3 - Participation at two Agency Hearings

As a third optional task, two senior URSGWC personnel will participate in two agency hearings. Some preparation for these hearings is also included.

SCHEDULE

As discussed with you, we anticipate that this work, through submission of a draft report, will be completed in 3 to 4 months. Modifying the report in response to comments is expected to require 1 to 3 weeks, depending on the number and extent of comments.

CLOSING

We will be happy to discuss this proposed scope of work with you at your convenience. Thank you for including us in your team for this interesting project.

Sincerely,

URS Greiner Woodward Clyde

Said Salah-Mars

Said Salah-Mars, Ph.D., P.E.
Senior Project Manager

Michael Stuhr for

Michael P. Stuhr, P.E.
Project Manager

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URS Greiner Woodward Clyde
A Division of URS Corporation

500 12th Street, Suite 200
Oakland, CA 94607-4014
Tel: 510.893.3600
Fax: 510.874.3268
Offices Worldwide

September 13, 1999

Ms. Aimee Dour-Smith
Jones and Stokes Associates, Inc.
2699 V Street
Sacramento, CA 95818-1914

Subject: **Report on Task 1 - Review Existing Data and Solutions, and Revise Scope of Later Tasks as Needed. Geotechnical Services in Support of Supplemental EIR/EIS, Delta Wetlands Project, California**

Dear Ms. Dour-Smith:

This letter report presents the results of our Task 1 of the subject geotechnical services. It contains the results of our review, responses to comments prepared by Hultgren-Tillis Engineers (HTE) on our proposed scope of work in the later tasks, and our recommendations for changes in some of the later tasks based on the review and comments.

1.0 INTRODUCTION

The following text is organized according to the sections in our Scope of Services dated July 6, 1999. Typically the text first provides our "Background" of a scope item from the July letter, then the specific task description from the July letter, followed by HTE' comments, and finally by our response to the comments and conclusions on changes in the scope of work where deemed warranted. In two scope sections (2.1 and 3.0), HTE also provided comments on our background statements, and these comments are also responded to.

We would like to note that the "tasks" numbered by HTE were actually bulleted scope items in our Scope of Services, and in this sense represented work items within a task rather than stand-alone tasks. Because HTE discussed them as tasks, we have adopted this format for this review.

We also note that on many tasks (as defined above), HTE provided no comments, rather only gave citations. We have reviewed all citations and noted where they will be of assistance.

2.0 PERFORM SEEPAGE ANALYSES

2.1 INTERCEPTOR WELLS

Background Information:

Active interceptor or relief wells are proposed for mitigating potential seepage impact on the neighboring islands as a result of filling the reservoir islands (Webb Tract and Bacon Island). Field groundwater drawdown programs were conducted by HLA on McDonald Island in 1989-1990 (Phase I) and 1990 (Phase II). McDonald Island is located adjacent to Mildred Island that was flooded at the time of the demonstration. The field test was conducted to evaluate the feasibility and effectiveness of the interceptor (relief) wells in lowering the hydraulic head in the sand aquifer. HLA (1991) also performed groundwater numerical modeling to simulate various systems of interceptor wells and the required rate of discharge (groundwater withdrawal) that would maintain the existing groundwater conditions at the neighboring islands.

An independent evaluation of the effectiveness and the active interceptor wells will be conducted to provide response to the SWRCB concerns about their adequacy. The activities proposed under this task will include:

HTE Comment: Our assessment of the November 25, 1998 letter form the State Board is that hearing participants were not convinced that a pumped well system could reliably control any groundwater seepage. We are confident that as a groundwater flow control mechanism, wells can control seepage. We believe URSGWC should review the concerns presented in the testimony to check that the concerns have been fully addressed.

URSGWC Response: Based on our review of the reference documents and the testimony cited above, we believe that the concerns raised by SWRCB are not fully addressed and supplemental analyses addressing specifically these concerns are needed. The justification is further developed in our responses to the comments in the following paragraphs.

2.1.1 Review the test data and conclusions made for the field drawdown program for use as a calibration to the numerical seepage model,

HTE Comment: The McDonald Island demonstration was not intended to be a basis of calibration. The nature of the recharge is not precisely known. McDonald Island landowners reported increased seepage impacts following the inundation of Mildred Island. However, some of the reported effects may be related to local deep dredging for levee construction materials that may have occurred in Latham Slough immediately adjacent to the levee.

For the Demonstration Test pumping, the fifteen wells were interconnected with a sealed header connected to a single pump. As a result, the individual drawdown and flow rate of each well is not known for the pumped test. The drawdown and flow rates were recorded for the gravity flow test. The key output from the McDonald Island drawdown tests are the changes that occurred in the hydraulic grade lines as measured by the piezometers.

URSGWC Response: The purpose of this task is to run a 2-D computer model with the same geometry and input conditions as those during the drawdown test to check what material permeability best fits the groundwater table measured in McDonald Island during the test. Gradation curves can than be compared to other island aquifers to make necessary adjustments to the permeability values. We consequently, recommend pursuing this analysis because it will help provide a higher comfort level in the material permeabilities.

2.1.2 Develop a baseline condition for the groundwater at the project site (part of Task 1 scope of work). This baseline condition represents the existing groundwater and seepage condition before the installation of interceptor wells and will be used to measure the effectiveness of any proposed well system.

HTE Comment: The three "groundwater" baseline data points that are available include (a) the mean tidal level in the slough or river; (b) the groundwater level measured by monitoring wells in the aquifer immediately below the levee; and (c) the groundwater level beneath interior portions of the island. This last point is not supported by existing monitoring wells and must be estimated. The groundwater levels in the aquifer below the perimeter levees are presented in the groundwater monitoring reports. What we found was that the groundwater level beneath perimeter levees was typically within 5 feet (above or below) the ground surface elevation at the levee toe (HTA 1992c, p. 3).

URSGWC Response: Mr. Hultgren provided insightful information on the baseline condition of the groundwater. Particularly the reference on groundwater monitoring program dated 1995c is very useful. The data from that report will be used in our subsequent analyses. A reduced effort is anticipated for this task.

2.1.3 Develop numerical models and analytical procedures for the groundwater withdrawal simulation.

HTE Comment: None.

URSGWC Response: The task on the numerical modeling of the groundwater withdrawal simulation will be conducted because it will allow the assessment of the potential impact on the neighboring islands' background groundwater (to assess the no-net change condition). The review of the references cited in HTE's comments indicates a very simplified model (1-D) was used which does not fully characterize the seepage through the silt and peat layer into the aquifer in the reservoir island, and out of the top layer in the neighboring islands.

2.1.4 Reconcile soil and groundwater parameters used in analyses with the field data (calibration)

<u>HTE Comment:</u> (Also see 2.1.1 and 2.1.2) The range of permeabilities in the sand can be estimated from grain size analyses. These are available in the preliminary geotechnical investigation and groundwater monitoring reports. Permeability of peat was measured for the Wilkerson Dam investigation. Infiltration from adjacent waterways is affected by past dredging; subsequent sedimentation; the nature of the original material beneath the slough, river or man-made cut; evapo-transpiration extraction; and irrigation practices.

<u>URSGWC Response:</u> This task will be rolled into task 2.1.3. If the channel stages are known along with the piezometers data in project and neighboring islands, some level of model validation should be tested before the production runs are launched (reduced scope).

2.1.5 Perform sensitivity analyses.

<u>HTE Comment:</u> None.

<u>URSGWC Response:</u> This task will consider applying a reasonable range of variation to the aquifer transmissibility (thickness and permeability) as reported in the various boring and wells logs. This task will be rolled into or combined with task 2.1.7 to minimize effort in analysis.

2.1.6 Evaluate the interceptor well system (diameter, spacing, depth, screened length) proposed by DW to maintain the existing groundwater condition at the affected islands during high storage in the reservoir islands, and develop recommendations for an optimal system,

<u>HTE Comment:</u> DW has not selected final diameter or spacing of interceptor wells. In making preliminary evaluations for cost estimates, we assumed typical spacings of 150 to 160 feet. We assumed 12-inch diameter wells with 6-inch diameter screened casings. We further assumed that the aquifer will be fully screened.

<u>URSGWC Response:</u> Because some concerns were raised regarding the adequacy of the pumping wells, this task will be an evaluation of DW's proposed well configuration and will determine the sensitivity of the pumping wells system to well spacing, diameter, and pumping rate. Based on the findings from the analyses, mitigation measures will be recommended as appropriate. We will assume that the wells are screened within the aquifer.

2.1.7 Address the variation in subsurface soil conditions at the project site and its effects on the interceptor well system.

HTE Comment: See comments in 2.1.4.

URSGWC Response: Task merged into 2.1.5.

2.2 EFFECTIVENESS OF MONITORING SYSTEM AND PROCEDURES

Background Information:

HLA developed a monitoring system for groundwater seepage. The monitoring system provides a standard of performance against which project related seepage can be determined.

2.2.1 and 2.2.2 Review the proposed standard monitoring procedures developed for the Delta Wetlands project, and assess the adequacy and effectiveness of the proposed procedures to monitor the project related seepage to the neighboring islands.

HTE Comment: None.

URSGWC Response: In this task we propose to assess the adequacy of the effectiveness of the monitoring program and provide recommendations if deemed necessary. Some of these recommendations may be derived from the model analyses proposed in tasks 2.1. In evaluating this task, we have reviewed the list of references provided by Mr. Hultgren and noted where the existing information will be of assistance in our analysis.

2.2.3 Determine volume and time-dependent variation of seepage under various groundwater and subsurface soil conditions (sensitivity analyses).

HTE Comment: (A) in our assessment, neighbors are impacted by changes in the elevation of the groundwater, regardless of flow quantity. (B) We do not understand the significance of doing time-dependent evaluations.

URSGWC Response: This task is mainly related to the issue of whether a time-delay exists between the stage filling of the reservoir and the increased piezometric head in the neighboring islands and the capacity of the pumping wells to relieve the excess head in the aquifer in due time. The outcome of this task may impact the rate of filling of the reservoir and also impact the interpretation of the monitoring program.

2.2.4 Evaluate the monitoring procedure proposed by DW using results of the sensitivity analyses. The existing monitoring procedures may be used and expanded to incorporate analysis findings

HTE Comment: None.

URSGWC Response: This task will be rolled into Task 2.2.2. We will evaluate changes to the proposed monitoring procedures as deemed needed.

2.2.5 Evaluate the criteria (termed "significance standard") developed by DW to determine whether seepage onto neighboring islands merited action by DW. Develop alternate criteria as needed, including easily usable tools such as plots and/or tables of correlation among various groundwater parameters, and set thresholds for different levels of response, including reporting to various agencies and the needs for emergency response.

HTE Comment: The significance standard was developed with the Seepage Committee who agreed to the final criteria.

URSGWC Response: This task will focus on evaluating how the proposed significance criteria translates into changes in groundwater conditions from the no-project conditions.

2.3 ROUTINE MAINTENANCE OF INTERCEPTOR WELLS

Background Information:

The SWQCB has expressed concern about the long-term reliability of the proposed extensive groundwater pumping, especially in view of some difficulties reportedly encountered during DW's demonstration project.

2.3.1 Evaluate the long-term reliability of the selected well system including its power supply.

HTE Comment: The McDonald Island demonstration project provided very useful and reliable information, however, when assessing the long term reliability of wells, limited weight should be put on the McDonald Island demonstration project. These wells were put in with the single purpose of conducting a short term test. We did not focus on filter pack gradation or well development. The permanent wells will be designed, installed and developed with the goal to be efficient and for long-term reliability. They also will be regularly maintained.

URSGWC Response: We agree with HTE's comments on the purpose and objective of the McDonald Island Test. Our input on this task will consist of evaluating the feasibility and long term viability of the proposed well system and, as needed, providing recommendations and general guide specifications to mitigate for inadequacies in the proposed installation and operation of permanent dewatering wells.

2.3.2 and 2.3.3 Estimate, using the models developed in Task 2.1, the effects of various plausible pumping outages, and develop routine monitoring procedures to identify and respond to outages or lack of performance of individual wells, well groups or the entire system.

<u>HTE Comment:</u> The obvious place to monitor "reportable" data is at the receptor (neighboring island). We believe no reportable monitoring beyond what we have already recommended is needed. DW will have additional monitoring and maintenance systems to keep their interceptor wells working effectively.

<u>URSGWC Response:</u> This task responds to the concern about potential outage of some or all pumping wells. This evaluation will identify the potential impact on the groundwater in neighboring islands from wells outage, and, as needed, recommend changes to the proposed monitoring program to respond to outages.

2.3.4 Develop routine maintenance procedures/guidelines for the selected system.

<u>HTE Comment:</u> Developing routine maintenance procedures will be part of final design.

<u>URSGWC Response:</u> Task deleted.

2.4 'UNAUTHORIZED' WATER DIVERSION INTO THE STORAGE ISLANDS THROUGH SEEPAGE

<u>Background Information:</u>

The SWRCB is concerned that during certain water level conditions in the storage islands and the adjacent channels the pumping from the interceptor wells or direct seepage may constitute "unauthorized" water diversion into the storage islands. A methodology is needed to prevent or account for such unauthorized diversions. We propose to conduct the following analyses to assess this potential impact.

<u>HTE Comment:</u> None

<u>URSGWC Response:</u> None. Scope is not changed.

2.5 EFFECTS OF BORROW PITS ON SEEPAGE

<u>Background Information:</u>

The project proposes to utilize borrow material from the storage islands to strengthen the island's levees. The SWRCB has expressed concern that the limitations on locations of borrow pits proposed by DW may not be adequate to prevent excessive seepage increases in the underlying sand aquifer due to the borrow pits. We propose to perform the following seepage analyses to assess this condition.

2.5.1 Assess the feasibility and effects of borrow pits on the seepage conditions using seepage numerical models developed in Task 2.1.

HTE Comment: *At the time of our 1991 interceptor well study (HLA 1991B), the project envisioned long and wide borrow pits to develop the large quantity of fill material for the spending beaches. The current project envisions using fill quantities associated with recognized levee upgrading practices such as Bulletin 192-82. The fact that borrow pit proximity to perimeter levees impacts well spacing and pumping rates is already established. Borrow pit placement will be selected during final design, considering depth of overburden, haul road locations, and proximity to perimeter levees.*

URSGWC Response: The scope of Section 2.5 will be condensed into one task. The two cases analyzed one-dimensionally by HTE will be used; however, further variations of width, excavation geometry and optimum location with respect to the reservoir levee will be identified and analyzed.

2.5.2 Evaluate the proposed size, depth, and setback locations of borrow pits, and make recommendations on an optimal system.

HTE Comment: *None*

URSGWC Response: May be optional depending on the results of the analyses of Task 2.5.1.

2.6 SETTLEMENT CAUSED BY FILLING AND PUMPING OF WATER

Background Information:

Rapid changes in the reservoir water level cause additional stresses on underlying soil layers. Groundwater pumping from under the levees also causes additional soil stresses. Both of these factors may cause additional settlements of levees and interiors of both storage islands and adjacent islands. This issue appears not to have been addressed in detail. For levee design as well as overall impacts on the project, such settlements should be addressed.

HTE Comment: *None*

URSGWC Response: None. Scope is not changed.

3.0 LEVEE STABILITY

Background Information

HLA (1989, 1992) performed geotechnical investigations and engineering analyses for the Delta Wetland levees. The study included field investigation, soil laboratory testing, analyses of levee stability, settlement and seepage through and under the levees.

As part of this study, HLA also developed site-specific static and dynamic soil properties by conducting geophysical surveys and laboratory testing. In addition, HLA developed seismic design load criteria and performed one-dimensional site response analyses. Liquefaction potential evaluation and seismic-induced deformation analyses were also performed.

More recently (1998), probabilistic seismic hazard analysis and levee failure probabilistic evaluation were conducted for the Sacramento/San Joaquin Delta levees by the Seismic Vulnerability Sub-Team of CALFED's Levees and Channels Technical Team. In this study, the delta region was divided into four groups based on their expected seismic ground motions and the levee fragility to failure. Estimates for levee failure due to scenario earthquake events from nearby dominant seismic sources were also developed.

The SWRCB identified various issues associated with the stability of the Delta Levees which included subsidence, static and seismic stability and deformation, settlement, erosion, and overtopping. Although additional work addressing these issues was not requested in the November 25, 1998 letter, during subsequent meetings between the lead agencies and the engineers it was decided that additional engineering analysis of these items was required. Accordingly, we propose to perform the following activities to verify compliance with the state regulatory agencies on reservoir stability issues.

HTE Comments:

Levee Stability Review Criteria - The approach taken by URSGWC applies to dams; that is, water retention structures under the jurisdiction of DSOD. In their opening paragraph, the reviewer states that "design criteria for levees were also prepared for DSOD permit approval." This is not correct. Nothing we did for "levees" was ever intended to be under the jurisdiction of DSOD. The reviewer may be confusing the Wilkerson Dam work with what we did for levees.

We conducted our analysis of the reservoir project at the maximum pool elevation of +6 feet to evaluate the worst case conditions. It is possible that once the project is permitted and enters the *final design stage that other requirements may dictate a reduced pool elevation. One of the technical factors that may limit pool elevation is wave runup during high wind events.*

Before URSGWC begins Phase 2 tasks, DW will review the Phase I work taking into consideration the costs and feasibility of storage at a +6 elevation. DW has always expected to make a decision at the time of final design as to whether to build the reservoirs to +6, but as a result of the REIRIS and the levee concerns raised by the SWRCB November 25th letter it appears it may be necessary to make that decision at the time of approval of the Phase 2 tasks.

<u>CEQA Evaluation:</u> In their November 25, 1998 letter, the SWRCB makes the point that "an inadequately constructed, maintained, or protected reservoir levee could suddenly crack or gradually erode, causing damage to property and neighboring islands." Perhaps a helpful restatement might be that existing levees in their current state of construction, maintenance or protections are at a higher risk of suddenly cracking or gradually eroding and causing damage to property and neighboring islands under current practices than will be for a methodically designed and carefully constructed reservoir island levee.

All of the investigative and preliminary design work described herein and in other documents which are a part of the SWRCB hearing record constitute what we consider preliminary investigation and design work. The level of rigor was directed to satisfy CEQA requirements and additionally provide the project proponent with enough design information that a reasonable economic analysis could be developed.

For the CEQA and preliminary design stages, our studies were made with the concept of comparing proposed levees with conditions that currently exist and will exist without the project. HE 1995a describes the overall benefits of the project compared to current alternatives. The project intends to substantially improve the existing levees by widening crests, flattening slopes and improving erosion protection. The preliminary analysis for the levees indicates that the reservoir levees should be more reliable when compared to existing levees. Further, the reservoir levees should be better able to withstand flooding should a section of levee fall. With its interior erosion protection, groundwater control system and export pumping system in place, the consequences of a breach occurring at a reservoir island are much less than for a farmed island without these facilities.

In the cumulative impact analysis of the no-project condition, conditions will exist many years from now if farming practice continues on the islands that need to be considered. The islands currently subside at 2 to 3 inches per year from farming practices. This means that 30 years from now the total effective height of the levees will be 5 to 7.5 feet greater than current conditions. Levee stability and reliability on farmed islands underlain with peat soil will tend to decrease with time and seepage will increase with time as the islands subside. Switching the land use to a water storage project will greatly decrease subsidence. The project has performed the level of analysis deemed appropriate to evaluate CEQA level impacts on perimeter levees.

Seismic issues: *We believe the recently completed CALFED Seismic Vulnerability of the Sacramento-San Joaquin Delta Levees (December 1998) will provide sufficient input for CEQA evaluations.*

URSGWC Response

Levee Stability Review Criteria - If the storage volume exceeds 50 acre-feet and the retention structure is higher than 6 feet, the reservoir falls under DSOD jurisdiction as we understand the criteria. Moreover, the SWRCB has indicated that they would consider requiring Delta Wetlands to obtain DSOD evaluation regardless of statutory requirements. Considering these facts, it is our judgment that the feasibility evaluation of Delta Wetland's levee designs requires at least a preliminary evaluation according to DSOD criteria.

CEQA Evaluations - We recognize that, in the most limited interpretation, CEQA requires a comparison between the present condition and the future with-project condition. In the context of the project proposed by DW, more specific information is needed on future impacts, both to better define the proposed project, its specific impacts and costs, and thereby to judge the broader impacts and implications, including feasibility of the project. Specific issues that we consider as needing additional evaluation at this time, without waiting for final design, include, based on our review of the project documentation:

- more information on planned construction sequence and time history
- more specific documentation of stability analyses, including soil strength parameters, critical failure surfaces, factors of safety
- absolute stability factors of safety, not just relative numbers
- more information on time effects, including estimated time history of construction, time history of dissipation of excess pore pressures and settlement, and time history of factors of safety
- more specifics on development of required quantity of borrow material

Seismic Issues - We do not agree that earthquake stability is adequately addressed by the general comparisons to earthquake reliability presented in the draft EIR/EIS and the reference to CALFED's probabilistic study of seismic vulnerability of the Delta levees of December 1998. A project of this significance commonly requires at least preliminary site-specific considerations, analyses and conclusions on seismic stability and seismic impacts already in the EIR/EIS stage.

Summary of 3.0 Levee Stability

We conclude that the uncertainties and gaps in the existing DW documentation on slope stability are such that the additional studies recommended in the Scope of Work are still needed, and recommend that they be implemented.

Additional responses only on those specific items where HTE had comments follow. Those scope items in the URSGWC SOW not commented on by HTE require no response or changes to the proposed scope.

3.1 LEVEE STRENGTHENING

3.1.2 Evaluate analysis results from previous studies on levee stability, including soil engineering parameters used.

HTE Comment: Undrained strength parameters for peat and soft clay were measured for Wilkerson Dam study on Bouldin Island.

URSGWC Response: The results of these studies on Bouldin Island are valuable, yet must be extrapolated to other islands judiciously.

3.1.4 Update dynamic soil parameters based on recent findings.

HTE Comment: Site specific studies are not needed. The EIR/EIS preparer should rely on the CALFED Seismic Vulnerability of the Sacramento-San Joaquin Delta Levees (December 1998) as the final source document for seismic reliability.

URSGWC Response: As noted under Section 3.0, we disagree with the concept that a Delta-wide, programmatic study would be an adequate basis for the EIR/EIS-level assessment of seismic risk.

3.1.7 Evaluate analyses done by DW of the safety of the strengthened levees against static and earthquake-induced loads. Implement additional analyses as deemed needed. Failure mechanisms should include slope failures, inadequate bearing capacity, excessive slope deformation and settlement, critical seepage conditions, piping and internal erosion around well screens, and others.

HTE Comment: Need to compare against existing conditions per CEQA.

URSGWC Response: See Section 3.0 URSGWC discussion regarding details of analysis required.

3.1.8 Evaluate the during-construction, long-term and rapid drawdown static stability of the levee systems, and compare the stability parameters to existing conditions.

HTE Comment: *Analyses previously performed by HLA. In considering a rapid drawdown condition, URSGWC should be aware that the reservoirs would be drawn down at a rate of up to 12 inches per day.*

URSGWC Response: Working with Jones & Stokes Associates we will make determinations on the maximum daily rate. Whether 12 or 18 inches, these rates are high enough that a rapid drawdown condition should be considered.

3.1.12 Address erosion by wind fetch and wave runup.

HTE Comment: *DW has an initial internal study for soil cement and riprap on various slopes. Documents can be made available.*

URSGWC Response: We will certainly review DW's additional documentation as a part of our study.

3.1.13 Address constructability of selected levee system. We will evaluate the gradation of the materials used to strengthen the levees (see Task II.5 for borrow pits).

HTE Comment: *Final system not selected. Likely to be similar to Bulletin 192-82 but with a wider crest.*

URSGWC Response: Issue addressed here is constructability, not selected levee system. The need to address constructability (i.e., staging of construction, lag time to allow consolidation between stages, and total construction duration) is discussed in our discussion in Section 3.0.

3.1.14 Review with DSOD

HTE Comment: *See Comment 3.0*

URSGWC Response: See Section 3.0, URSGWC Response.

Summary of 3.1 Levee Strengthening

HTE's comments explain DW's approach and position, but result in no significant change, in our opinion, on the scope of work proposed under task 3.0. Accordingly, we recommend that the scope of this task remain as proposed; we will consider HTE's comments and references when conducting the analysis.

3.2 ASSESS EFFECTS OF INTERCEPTOR WELLS ON LEVEE STABILITY

There were no HTE comments on this section and no change in the proposed scope is considered. However, the lead agency has requested that, in addition to the previous variables to be addressed regarding levee stability, the effect of potential internal erosion and piping around well screens be evaluated and, as needed, mitigation or monitoring measures be recommended.

4.1 OPTIONAL TASK 1 - ASSESS POTENTIAL DAMAGE TO NEIGHBORING ISLANDS IN EVENT OF LEVEE BREACH OR PROJECT ABANDONMENT

Background Information:

The SWQCB has expressed concern about potential damages to adjacent islands in the event of a levee failure of a storage island and in the event of project abandonment by the owner. Some effort to address these concerns, using various plausible scenarios, appears justified.

4.1.1 Formulate scenarios of levee failure of storage islands that might damage adjacent islands; one example is levee failure with full storage island (elevation +6 feet) and extremely low water in the channels.

HTE Comment: Other examples should be no-project alternate during both Delta flood and Delta drought conditions.

URSGWC Response: The no-project alternative can be considered. While this alternative is valid in a relative sense (comparing with-project to without-project), it does not address the absolute potential effects of the project, which we believe is needed, as indicated in Section 3.0.

4.1.2 Estimate potential effects of these events on levee across a narrow channel, and judge the likelihood of any catastrophic damages (some erosion damage could easily be repaired after the event).

HTE Comment: Should also consider no-project alternate.

URSGWC Response: Same as response to 4.1.1.

4.1.3 Formulate scenarios of levee abandonment at critical times in the storage islands' annual use cycle, or after a damaging event such as an earthquake (but that does not cause a levee failure).

HTE Comment: Scenarios should also consider abandonment when farming became uneconomical (no-project alternate).

URSGWC Response: Essentially same as response to 4.1.1.

4.1.4 Estimate storage islands' behavior for these scenarios, and seepage conditions that could negatively impact adjacent islands; estimate the potential for significant short- and long-range damages to adjacent islands.

HTE Comment: Reviewer should also address beneficial impacts of an abandoned storage island.

URSGWC Response: Essentially same as response to 4.1.1

4.1.5 Work with Jones and Stokes' hydraulic modelers to estimate the probability that these conditions could happen.

HTE Comment: Abandonment will be an economic event. It will also be a function of ownership.

URSGWC Response: Clearly the nature and stability of the ownership affect the probability of abandonment, and will be considered.

Summary of Optional Task 4.1

HTE's comments, if implemented, would broaden the scope of the review slightly, but do not eliminate any of the proposed scope of work. Accordingly, we recommend that the proposed scope of Option 1 be implemented essentially as proposed, considering HTE's comments as appropriate.

Please call with any comments.

Sincerely,

URS GREINER WOODWARD CLYDE

Said Salah-Mars

Said Salah-Mars, Ph.D., P.E.
Senior Project Manager

Michael Stuhr for

Michael Stuhr, P.E.
Program Manager

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